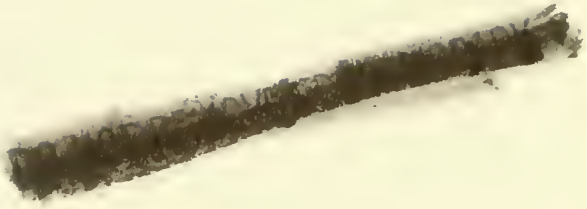

TRANSACTIONS

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AMERICAN SOCIETY

OF

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(INSTITUTED 1852.)

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ERRATA.

Vol. XIX, September, 1888, page 109, eleventh line from the bottom, "page 13" should read "page 103."

NOTE.—In the Transactions for September, 1888, Vol. XIX, page 92, Table No. 2, and also on Plate XIII, page 124, reference is made to data from tables published by the Cincinnati and Newport Pipe Co. We are now informed that these tables were originally prepared by Messrs. A. H. Howland and George A. Ellis, Civil Engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

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393.
(Vol. XIX.—July, 1888.)

PRELIMINARY REPORT OF THE COMMITTEE ON THE PROPER RELATION TO EACH OTHER OF THE SECTIONS OF RAILWAY WHEELS AND RAILS.

To the President and Members of the American Society of Civil Engineers:

Your Committee to whom was referred the question raised in the subjoined resolution* have been greatly impeded by pressure of other duties from holding such meetings and giving the matter such study as would warrant the submission of a final report in the first instance.

* The Committee is acting under the following resolution:

"*Whereas*, The relation which the form of the head of a rail should bear to the section of a car-wheel tread and flange has recently been in dispute, it being asserted on the one hand, that they should have as long a line of contact as possible, and, on the other hand, that such long contact would be dangerous and injurious; and,

Whereas, The question raised by this diversity of view is of direct importance to the many members of this Society, as well as to the public interest, and hence is one which this Society may appropriately consider through Committee; therefore be it

Resolved, That a Committee of five members of this Society be appointed by the President to consider and report to the Society on the proper relation to each other of the sections of rails and wheels; to what extent and at what points it is expedient that their sections should be such as to bring them normally in contact, and to what extent and at what points it is not expedient that they should be so in contact; and that the Committee be instructed to seek information from all those who are known to have given the subject attention."

This resolution was submitted to letter ballot and adopted by a general vote of the Society. Subsequently, by vote at the annual meeting, the President was authorized to add two additional members to such Committee.

and in view of the importance of the question raised it seems undesirable to do so in any case until the information which they have collected and the conclusions which seem to be indicated have been submitted for discussion, with the probable result of bringing out further information bearing on the matter. They therefore submit the following preliminary or progress report, postponing to a later date the preparation of a final report giving conclusions.

Your Committee understand the question submitted to them to be in substance this: Is it preferable that the sections of rail and wheel should stand to each other in the relation shown in Fig. 1, in which the fillet of the flange is to a larger radius than the corner of the rail, so that the rail and wheel can only come in contact on the top or at a single point on the corner, or is it desirable that the radii of the flange-fillet and rail corner should be the same, as also the radii of the inside half of the rail-top and the corresponding part of the wheel-tread, so that the two shall be normally in contact with each other over the entire distance from *A* to *B*, both on curves and tangents, as shown in Fig. 2, whenever the flange approximates to the wheel at all.

The Claims Made on Each Side.

Summarizing these as impartially as may be, it is claimed in behalf of the form shown in Fig. 1 :

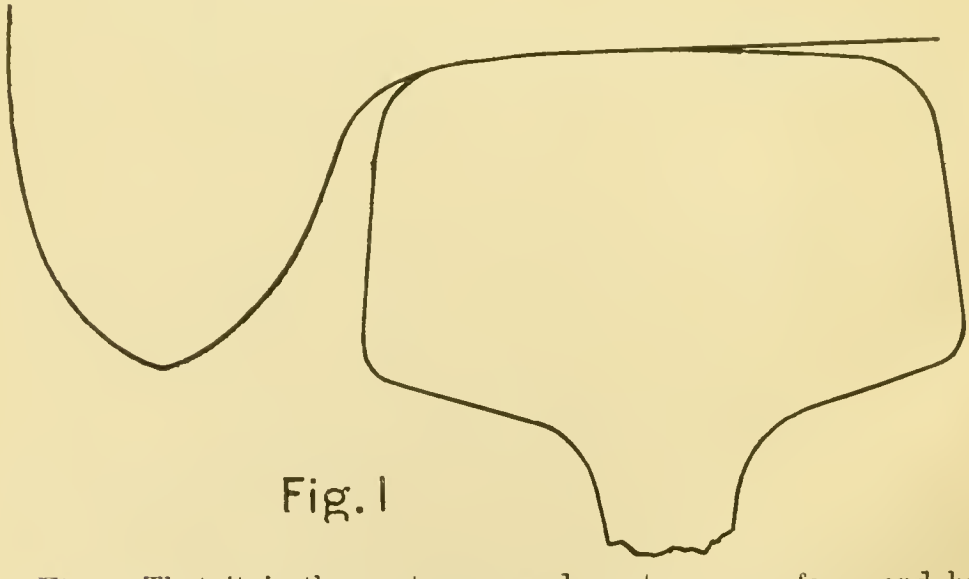

Fig. 1

First.—That it is the customary and most common form, and has the burden of proof in its favor, being now almost universal.

Second.—That it is the most practically convenient and attainable

form, since it does not require the radii of either fillet or rail corner to be always the same, whereas the conditions of Fig. 2 are only attainable by universal agreement to make both rails and wheel always of the same radii.

Third.—That it is the form of least wear, since on tangents and on the inside rail of curves there is a purely rolling contact on top, and on curves an almost purely rolling contact on the corner of the rail, whereas in Fig 2 there is rubbing flange friction at *A* both on curves and tangents, which produces an excessive wear from the beginning of the life of the rail, such as can only exist in Fig. 1 on old outside rails of curves after they have been much flange worn, as in Fig. 2 below.

Fourth.—That it is the safer form, in that the flange will not so readily mount the rail.

Fifth.—That the form of the rail is not as a matter of fact a cause for sharp flanges to any appreciable extent.

Sixth.—That even if it were, the loss by sharp flanges is a small one, if indeed there be any.

On behalf of the form shown in Fig. 2 it is claimed:

First.—That the form shown in Fig. 1 offers too little bearing surface, which it is desirable to increase.

Second.—That the form shown in Fig. 1 is a cause for sharp flanges, which is removed by making the fillet and rail corner exactly fit each other, as in Fig. 2.

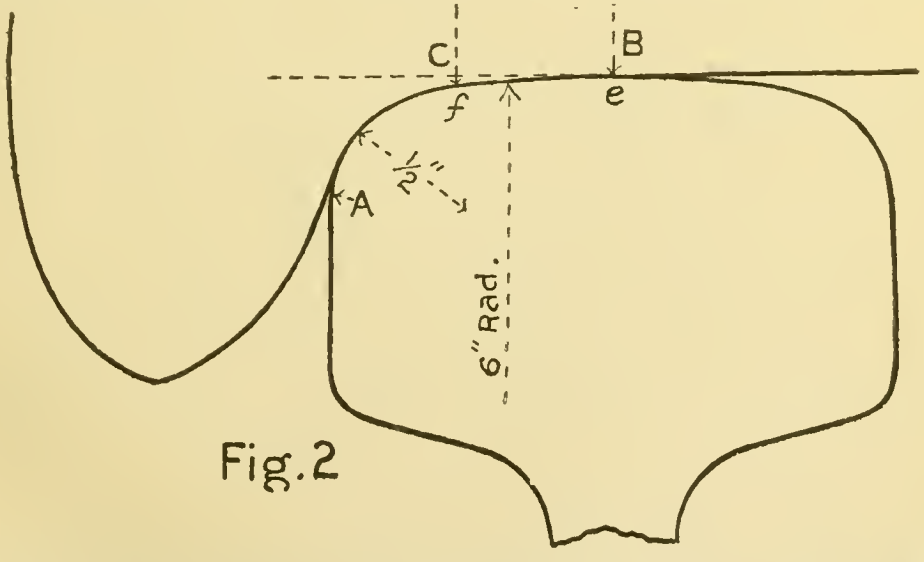

Fig. 2

Third.—That the loss from sharp flanges is so serious as to demand a remedy.

Fourth.—That no extra wear or extra danger of derailment will result from the form shown in Fig. 2 to outweigh its alleged advantages.

Your Committee have collected quite a body of statistical information relating to the question submitted, much more, they apprehend, than has ever been collected in relation to the question before, including some of a quite novel character specially compiled for them, which sheds a great deal of light on it. This is especially true of the records of the Pennsylvania Railroad, which from the many years (ten) and vast number of wheels which they cover, are of quite unequalled value for the purpose in view. Their completeness has also enabled your Committee to trace to its source an error of great magnitude and importance in regard to the proportion of sharp flanges on the Pennsylvania and other roads, which appears to have had a good deal of influence in creating the antagonism of views which led to the appointment of your Committee.

They have subdivided their investigation, and the following report, under the following heads: First, Historical; second, Statistical; third, Theoretical; fourth (to be given in a final report), Summary of conclusions.

History of Action on this Question.

Your Committee have not been able to discover that the doctrine that the rail and fillet radius should correspond has ever been advanced in any authoritative way until within the past few years. The first formal action to that effect was a brief report by a Committee of the Master Car Builders' Association in 1883, who stated that they were "strongly of opinion that by far the greatest cause of sharp flanges" was the sharp corners of the rails, and recommended that the rail be rounded off to a $\frac{5}{8}$-inch radius, so as to "exactly fit the flange." The conclusions of a previous Committee, that sharp flanges were due to special defects of truck and wheels, bad gauge of wheels, trucks not square, wheels not of the same diameter, overloading of side bearings, unequal loading of car, imperfect chilling, etc., were enumerated as secondary causes.

No action beyond receiving it was had on this report, but immediately thereafter Mr. M. N. Forney, M. Am. Soc. C. E., Secretary of the Association, took up the matter in an elaborate paper presented at the 1884 Convention, information for which was gathered by circular in

the name of the Association and which was (in effect) endorsed and accepted by the above-mentioned Committee as giving their views and relieving them of the necessity of making a report.

This paper has been very widely circulated. Officers of Maintenance of Way were formally invited to attend and take part in its discussion, and your Committee have reason to believe that this paper and resulting action thereon has largely given rise to the unfounded impression, which is nevertheless widespread, that the Master Car Builders' Association, as a body, has acted on this matter, and that the paper was *a quasi* official protest on their part against the bad form of rails. This is not the case.

At about the date of this paper the Pennsylvania Railroad changed their standard to a round cornered rail, and one or two other roads followed suit. Several years previously, in 1883, the Lehigh Valley Railroad adopted what is now their standard section, and generally known as the Sayre rail, designed by Mr. Robert H. Sayre, C. E. In regard to his reasons therefor he states in a letter to a member of your Committee:

"The sharp cornered rail attacks the wheel in its most valuable point, and while the rail is being worn down to conform to the normal shape of the wheel, you are ruining the wheel, which costs more per ton than the rail, by cutting through the chill and wearing the flange straight. Carefully kept records on the Pennsylvania Railroad, for a period of five years, while they used a sharp cornered rail section, showed that 40 per cent of their wheels removed were worn through the chill in throat or had straight flanges. I had records kept on the Lehigh Valley Railroad for three years, and found the loss from the same cause was less than 4 per cent."

Passing for the present the questions raised in the first part of this quotation,* the substance of the closing sentence has been repeatedly alleged by various persons in the discussions of the Master Car Build-

* It may perhaps be well that your Committee should here note, however, that the first sentence above, in respect to "the rail being worn down to conform to the normal shape of the wheel" does not seem to conform to the facts of experience, as evidenced by the accompanying sections of worn rails (Figs. 32 and 33), reproduced from Mr. Forney's paper, before referred to. Fig. 32 is from a rail laid on tangent on the Philadelphia and Reading Railroad in 1875, and taken up in 1882, after about 105,000,000 tons had passed over it. Fig. 33 is from a similar rail laid in 1867, after 176,000,000 tons had passed over it. In both cases it will be seen that the original round corner has worn down to a very sharp corner. It is obvious that if it be in fact the case that the rail corner tends to cause sharp flanges by wearing away the fillet to a smaller radius, the tendency of the flange fillet should be to wear off the rail corner to a larger radius; yet the two rails shown, made with the original corner-radius nearly as large as the flange fillets, for the express purpose of avoiding wear in the latter, have worn down to about as small corner radii as are ever used in new rail sections.

ers' Association and elsewhere, and your Committee think there is reason to believe that this one supposed contrast in statistics has had very great influence with many persons and on much of the action had, in favor of round-cornered rails to fit the flange. They have therefore investigated its truth with all possible care, and find that it is founded on an entire misapprehension, the error being that the per cent. of worn-out rails only on the Pennsylvania is compared with the per cent. of all wheels on the Lehigh Valley. The correct figures, which appear in this report, and which have never been in print or even collected in manuscript before, indicate that the Lehigh Valley percentage is not now and

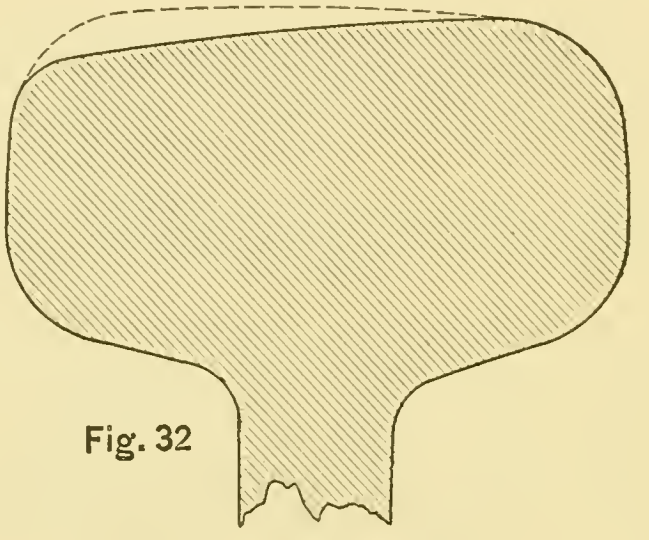

Fig. 32

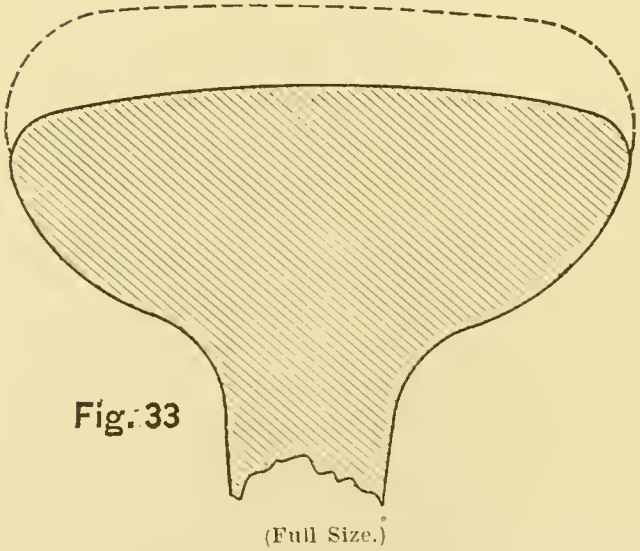

Fig. 33

(Full Size.)

Worn rails, Philadelphia and Reading Railroad, showing tendency of round-cornered rails to wear sharper.

has never been lower than on the Pennsylvania, and (apparently) that on both roads the percentage of sharp flanges has increased very materially since the adoption of the round-cornered sections.

The paper by Mr. M. N. Forney, before referred to, while quite elaborate in other ways, bases the conclusion that the radii of rail and fillet should correspond on the following argument only :

"As the maximum weight carried by car-wheels is now from 5 000 to 8 000 pounds, the bearing surface must be subjected to pressures of from 40 000 to 64 000 pounds per square inch. It is therefore not surprising that they are rapidly worn away, as there is no principle in mechanics more firmly established or more certain than that the wear of surfaces in frictional or rolling contact is in an inverse proportion to their area. Therefore, if we should increase the area of the surfaces of the wheel and the rail which are in contact, the capacity of both for resisting wear will be increased. To do this their forms must be made to correspond to each other. In other words, the treads and flanges of wheels should be made of the same shape as the heads of the rails."

Postponing for the present further discussion of this argument than to say, first, that it neglects the question of whether the recommended course will not substitute mixed rolling and sliding friction for purely rolling contact, and so materially change the nature of the problem, and second, whether the pressures named are really injurious for the bearing surfaces of wheels and spheres, because they are undoubtedly so for the bearing surfaces of prisms, which have no greater section anywhere than their bearing surface, your Committee present it in full, as the only scientific argument which has ever been made public, to their knowledge, in favor of a doctrine which has obtained such wide currency as to be acted on practically by several of our leading lines. Your Committee now proceed to summarize the statistical facts which they have collected in regard to the wear of rails and wheels as affected by their forms.

STATISTICAL RECORDS.—THE PERCENTAGE ON SHARP FLANGED WHEELS.

On this question, as also on the far more directly important question of comparative mileage life, your Committee have collected the information, which appears in the accompanying tables and subjoined appendices, being as much as seemed at all necessary for the purpose.

The relative percentage of sharp flanged wheels needs to be considered in connection with statistics of comparative mileage, to be shortly presented, and it is also to be remembered and allowed for that there is much irregularity in classification. This is especially conspicuous in steel-tired wheels. Thus, the Boston and Albany and Lake Shore and Michigan Southern report no sharp flanges whatever among steel-tired wheels. On the other hand, the Chicago, Burlington and Quincy reports 47 to 50 per cent., although the mileage of these sharp-flanged wheels is higher than any other.

The total per cent. of sharp flanges reported, out of wheels removed for all causes, may be said to range from 2 to 12 per cent. in passenger service, and from 3 to 20 per cent. in freight and engine service. The following are a few definite figures :

TABLE No. 1.

SUMMARY OF THE PERCENTAGE OF SHARP-FLANGED WHEELS IN THE TOTAL REMOVED ON VARIOUS RAILROADS.

ROAD.	Year.	Per Cent. Sharp Flange Wheels.	
		Passenger.	Freight.
Boston and Albany	1884	none.	2.97
Lake Shore and M. S	1886	12.2	no record.
" (steel tired wheels)	1886	none.	.....
Chicago, Burlington and Quincy	1885	6.4	21.5
" "	1886	10.8	20.4
" (steel tired wheels)	1885	47.3	.. ..
" "	1886	50.4	.....
New York, Lake Erie and Western	1884	....	5.8
Pennsylvania	1883	4.45	.....
Pennsylvania Company	1885–86	2.75*	5.0*
Lehigh Valley	1886	....	8.8

Many of the irregularities of this table, as notably for the Chicago, Burlington and Quincy, are due to peculiarities of classification, but further precision or a more elaborate summary appears unnecessary in view of the following facts as to relative mileage life.

* Estimated, by taking one-fourth of the per cent. of worn out wheels only, which is about the ratio of the Pennsylvania returns, which give both percentages.

The Relative Mileage Life of Sharp-Flanged Wheels.

Your Committee find it clearly indicated from all the data which they have collected, and in part append to this report, that the average mileage life of wheels removed for sharp flanges is very decidedly above the average life of all wheels, and is even, with one partial exception, the highest among the various causes for removal of wheels worn out by legitimate wear only. This is assuming the quality of the wheels to be of fair to good quality. As the quality of the wheel falls below this average quality (which includes, perhaps, three-quarters or seven-eighths of all the cast wheels in the United States), the statistics to be shortly presented will show that the percentage of sharp flanges very rapidly increases, and it may well be (although your Committee have no evidence of that fact) that the relative mileage life of sharp flange wheels is less, since bad wheels are particularly apt to be soft in the fillet chill.

The most valuable and decisive evidence as to relative mileage life are the returns of the Pennsylvania Railroad, shown in Table No. 3, and others, since they cover a long period of time (1878–87, ten years) and a large number of wheels, during which time a rigid, minute and uniform system of classification has obtained. The main part of this record covers only wheels which have failed by wear from legitimate causes, excluding all "slid flat" wheels (which covers from 50 to 60 per cent. of all removals under the rigid Pennsylvania inspection), as also all wheels removed for any cause fit for refitting or freight (20 to 25 per cent. of all removals), and also wheels removed as cracked or broken (less than 1 per cent. of the total). The average mileage of these excluded wheels is, of course, vastly lower than those which are included, which fail from wear.

Those which fail by legitimate wear only, are sub-classified under these eight heads : 1, "Shelled out;" 2, "Comby;" 3, "Seams;" and 4, "Worn flat;" terms which in a measure explain themselves, and imply more or less imperfection in the wheel; 5, "Worn flange," under which head are included all wheels ordinarily classed as sharp flanged; 6, "Hollow at flange," wheels worn in tread close to the flange, but not so as to cut into the flange abnormally; 7, "Hollow from flange," which means wheels worn deepest in the tread away from the flange, and presumably mated with wheels which run to flange. To these is added a very small class of miscellaneous.

The comparative service which wheels removed for each of these defects have rendered on the Pennsylvania Railroad for the past ten years is given in detail in the Appendix, and summarized in the following Table No. 2, the significance and importance of which, as respects the question now under discussion, your Committee think it would be difficult to exaggerate. They are not aware that any similar statistics have ever been computed or made public before.

TABLE No. 2.

RELATIVE MILEAGE LIFE of Worn out Wheels only (excluding Slid-Flat and Cracked and Broken Wheels) on the Pennsylvania Railroad. Average of Nine years, 1878-86, as shown in detail in Table No. 3.
(1887 added separately for comparison.)

CAUSES OF REMOVAL. (In inverse order of mileage.)	AVERAGE MILEAGE, 1878-86.			
	Baggage, Express and Postal.		Passenger, Combination and Emigrant.	
	1878-86.	1887.	1878-86.	1887.
1. Comby	62 120	70 369	53 840	60 924
2. Seams	64 880	88 974	52 150	62 916
3. Shelled out	71 290	75 483	54 980	58 237
4. Worn (not slid) flat	73 110	78 368	61 310	56 510
5. Hollow in tread at flange	75 550	59 866	63 810	37 516
6. Worn (sharp) flange	78 720	75 390	66 790	60 919
7. Hollow in tread from flange	81 810	70 866	67 450	50 504
Average of the two classes of wheels which fail from wear of tread only and not of flange (Nos. 5 and 7)	78 680	65 366	65 630	44 010
Against sharp flange wheels	78 720	75 390	66 790	60 919
Average of causes 1 to 4	67 850	78 298	55 652	59 647

TABLE No. 3.

SHOWING IN DETAIL THE AVERAGE MILEAGE of Wheels Removed as Worn Out, for each Leading Cause of Removal. Pennsylvania Railroad, 1878-86, inclusive. Wheels Removed as Cracked, Broken or Flat from sliding excluded.

ABSTRACTED FROM THE FULLER DETAILS OF TABLE A, APPENDIX.

1 = 1 000 miles.

BAGGAGE, EXPRESS AND POSTAL CARS.

	Shelled Out.	Comby.	Seams.	Worn Flat.	Worn Flange.	Hollow at Flange.	Hollow from Flange.
1878	88.64	73.20	88.70	77.13	83.05	74.05	91.13
1879	64.24	47.59	75.80	80.97	90.28	81.09	95.43
1880	74.65	53.59	69.35	86.75	85.51	79.78	93.32
1881	68.51	46.89	31.69	60.69	76.17	95.31	80.99
1882	63.98	61.01	48.70	66.13	74.84	70.42	71.35
1883	63.19	64.17	51.70	72.07	71.70	68.53	70.80
1884	65.59	68.22	68.57	62.53	78.06	58.85	75.43
1885	76.12	71.72	67.86	73.73	73.27	79.90	82.85
1886	76.73	72.64	81.59	77.98	75.37	72.06	75.01
Average.....	71.29	62.12	64.88	73.11	78.72	75.55	81.81

PASSENGER, COMBINATION AND EMIGRANT CARS.

1878	65.87	50 91	52.82	64.60	72.18	71.22	82.20
1879	61.29	46.63	87.39	69.25	79.85	77.32	81.38
1880	60 62	53.31	52.61	75.96	76.97	82.23	79.67
1881	59.02	44.61	23.20	57.14	62.51	76.02	71.00
1882	51.99	54.12	37.87	57.78	69.32	64.59	68.37
1883	45.67	54.36	44.56	52.24	67.72	49.90	58.43
1884	44.45	60.11	57.33	58.58	60.33	51.27	55.15
1885	50.49	58.75	56.72	56.28	56.70	54.77	57.29
1886	55.39	61.74	56.88	59.96	55.53	46.94	53.57
Average.....	54.98	53.84	52.15	61.31	66.79	63.81	67.45

This table shows only the nearest even ten miles, because the average was so computed to save space and time.

It will be seen that the average mileage of wheels with "worn flange" is very markedly above the average of other wheels which fail from wear, and is even appreciably higher than the average of the two select classes of choice wheels which fail from "old age" only, *i. e.*, those which show no other sign of defect up to their date of removal than that the tread is too deeply worn (although evenly worn all around) to permit of longer service. This class of wheels constitute only a very small percentage of the total drawn from service (2½ to 3 per cent.), just as old age constitutes a very rare cause of death among human beings, and for precisely the same reasons. It requires an entire absence of specific defects of structure.

An alleged disease among human beings which showed a somewhat higher average age at death than the average of deaths from old age, would not ordinarily be regarded as a very serious matter, nor would there be any great anxiety to decrease the number of those who died by it; and your committee cannot but regard it as a highly significant and important fact that in this most complete record available (which agrees in substance with all the others subjoined), the wheels which show the very highest mileage, practically, are those which have been drawn at last because they were suffering from a disease which it has been supposed was so important to cure, "worn flange."

This is especially notable, if we consider further that (1) there is much less metal to wear away to make a wheel sharp-flanged than to make it hollow-tread; (2) that a hollow-tread wheel has a much greater bearing surface than one which is running to flange, which should reduce the rate of abrasion if the greater bearing surface is a gain, and (3) and chiefly, that inspection will naturally be much sharper and removal quicker with sharp-flanged than with worn tread wheels, the latter being in no sense a dangerous defect.

The following Table No. 4, shows the percentage of the various causes of removal among worn out wheels only, from which it will be seen that the worn flanges are now 37 per cent. of such wheels, and the worn treads, including "worn flat," 13 per cent. The steady increase in number of sharp flanges is notable.

TABLE No. 4.

PERCENTAGE OF VARIOUS DEFECTS in wheels removed as worn out (excluding "slid flat" and "cracked and broken" wheels) from Passenger, Combination, Emigrant, Baggage, Express and Postal Cars, Pennsylvania Railroad.

FOR DETAILS, SEE TABLE A, APPENDIX.

CAUSES OF REMOVAL. (Worn Out Wheels only).	Per Cent. of Each.			Average Mileage Passenger, Combination and Emigrant.
	1878-83, Six years.	1884-86, Three years	1887, One year.	1878-86.
Shelled out	29.83	17.96	13.15	54 980
Comby	7.48	33.88	36.09	53 840
Seams	5.28	1.92	0.89	52 150
Worn flat	10.36	3.90	2.26	61 310
Worn flange	26.63	29.87	36.72	66 790
Hollow in tread at flange	4.84	2 73	1.87	63 810
Hollow from flange	11.90	9.08	8.42	67 450
Miscellaneous	3.68	0.66	0.60	.....
Total (worn out only)	100.00	100.00	100.00	.....

NOTE.—The round-cornered rail ($\frac{5}{8}$-inch radius) was made standard on the Pennsylvania in 1884, and in 1884 a change of fillet radius was made from $\frac{3}{4}$ to $\frac{5}{8}$-inch.

Of the wheels excluded from this table, "slid flat" wheels constituted (probably under a very rigorous classification) from 50 to 60 per cent. of the total removals; "cracked and broken," only a very small per cent.; considerably under one per cent.. Wheels fit for freight service are also excluded, and constitute 20 to 25 per cent. of the total removals.

The more important further evidence as to relative mileage life collected by your Committee, may be summarized as follows:

Pennsylvania Company, 1885–86—passenger wheels, 77 809 miles for sharp flanges, against 81 460 miles average of worn out wheels only. Freight wheels, 79.1 months against 90.5 months. Engine wheels, 83 296 against 83 983 average; all of worn out wheels only.

Lake Shore and Michigan Southern, 1886—Passenger, locomotive and tender cast wheels, 53 572 miles for sharp flanges, against 56 585 for other causes.

Chicago, Burlington and Quincy—

		Mileage.		Percentage Excess.
		Sharp flange.	Others.	Sharp flange.
Steel tired wheels, 33 inches....	1885	43 832	33 615	30.4
	1886	64 393	57 570	11.84
" " all sizes.....	1885	89 241	85 704	4.13
	1886	82 363	90 109	— 8.60
Cast passenger wheels, 33 inches.	1885	43 534	27 099	60.5
	1886	38 565	34 931	10.4
		Life in Months.		
Cast freight wheels, 33 inches..	1885	63	59	6.8
	1886	64	62	3.3

As further evidence, in the Master Car Builders' Report for 1881 a list of forty-two single wheels is given for another purpose, eight of which had sharp flanges, but the mileage of the sharp flanged wheels was considerably higher than the average of those removed for any other cause, even the general one, "worn out."

Your committee know of no evidence of a contrary character to the above to indicate that flange-cut wheels ever show a low mileage. On the contrary, it appears to be a fact well known to those who have given attention to the matter (although it does not seem always to have been duly considered), that sharp-flanged wheels make a very high mileage.

Mr. T. N. Ely, M. Am. Soc. C. E., General Superintendent of motive power of the Pennsylvania Railroad, says in a letter subjoined:

"The average mileage of wheels removed for all causes shows that there is no great difference between those removed for worn tread (or 'hollow from flange') and those removed on account of 'worn flange.'"

"In the majority of cases, however, these worn 'hollow from flange' give the higher mileage, although in some years the reverse was the case."

"In all cases, however, the average mileage of wheels, with these two defects, was much higher than for the other causes."

These notes appear to be based merely on comparison of the nine years separately. When averaged together, as Tables Nos. 2, 3 and 4 above, they give the still clearer and more emphatic evidence shown.

We abstract in Table No. 5 the percentages for each year of sharp flanged wheels (in the worn-out wheels only) on the Pennsylvania Railroad, which shows still more clearly that the percentage of sharp flanged wheels has been increasing since the round-cornered standard rail-sections of 1884 and 1885 were adopted, although that section was adopted with the express purpose of decreasing flange wear.

TABLE No. 5.

PERCENTAGES OF WORN OUT WHEELS only (Excluding Cracked and Broken and Slid Flat Wheels) Removed on the Pennsylvania Railroad, 1878 to 1887, for Worn or Hollow Flanges. [Altoona Cast Wheels, of substantially same composition and quality throughout the period covered.]

CLASS OF SERVICE.	1878.	1879.	1880.	1881.	1882.	1883.	Average.
Baggage, Express and Postal..	29.20	24.14	35.19	30.98	16.28	17.23	25.50
Passenger, Combined and Emigrant.....	28.57	25.45	27.84	34.03	27.95	22.08	27.75

CLASS OF SERVICE.	1884.	1885.	1886.	1887.	Average.
Baggage, Express and Postal.....	23.74	31.67	39.65	39.76	33.70
Passenger, Combined and Emigrant.....	22.32	28 05	33.79	33.69	29.46

In view of all the preceding, your Committee see no escape from the conclusion that in spite of the less bearing surface, the less material to wear away to condemn, and the closer inspection, wheels which are wearing sharp at flange make, if anything, a larger mileage than any other class of wheels, and certainly show no appreciably less mileage under normal conditions. This rather surprising conclusion, which was unexpected to every member of your committee, necessarily implies that the rate of abrasion of metal from wheels which are wearing sharp at flange must be materially less in pounds per thousand miles than in wheels which are wearing hollow in tread, because there is a smaller amount of metal to wear away to have the wheels unserviceable. We shall shortly see reasons why this should be so, both from theory and from observations on rail-wear, but before doing so it will be desirable to summarize the evidence of statistics as to the causes of sharp flanges.

The Causes of Sharp Flanges.

The following causes are all admitted to have an effect on sharp flanges, besides or in addition to, the effect of the form of the rail which is under discussion.

First.—Difference in diameter of wheels.

Second.—Trucks out of square.

Third.—Bad quality of wheels.

Bad gauging of wheels, unequal loading, etc., etc., are also admittedly aggravating causes.

While the existence of these defects is undoubted, often in an aggravated degree, yet there is a possibility which demands consideration, that they may produce sharp flanges when the rails are sharp cornered, and not produce them when the rail section fits the fillet of the flange. In view of the preceding conclusions as to the great comparative life of sharp-flanged wheels, if they be admitted to be correct, the question loses much of its importance, yet as those conclusions may not be admitted, it appears desirable to examine the probable effect of each of the above causes on sharp flanges.

Difference in Diameter of Wheels.

It is probable that few cast-iron wheels, mated on one axle, are exactly of the same diameter or exactly round. In not a few cases they are slightly out of center. In others they are of different hardness, so that if of originally the same diameter, they do not remain so. In others, again, the tread wears irregularly, so that the diameters are different in one position, although the same in another. In much less degree these conditions obtain with steel tired or dressed cast wheels. Great carelessness is often shown in mating wheels.

Mr. M. N. Forney, in the paper before referred to, demonstrates by results of experiments made with models, that even in a rectangular wheel-base, if the wheels are of different diameters, there is always a tendency to run in a curve. This tendency becomes very slight when the wheel-base is long, especially when the difference of diameter is small, but it always exists. Mr. A. M. Wellington, of your committee, has repeated these tests with a differently constructed apparatus, now in the possession of the Society, with the same result. If the wheels of only one axle are of different diameter, while the wheels of the other axle are of equal diameter, it is probable, although not demonstrated, that the tendency to run in a curve will be aggravated, for reasons which will be obvious on consideration.

Whenever there is the slightest difference in the diameter of the wheels, therefore, there will exist a tendency to roll toward one rail,

which must be continuously resisted by the flange; in other words, the flange reaction must continuously slide the wheels laterally on the rails. The power, or energy, wasted thereby will vary with the distance slidden through in a given time, and will ordinarily be trifling, but the flange pressure or reaction will not vary, since it takes the same pressure in pounds to slide a body an infinitessimal distance as to slide it a greater one; or even a greater pressure, because of the higher coefficient of friction.

There must, therefore, on both curves and tangents, be a large proportion of wheels which are exerting a very considerable flange pressure against the rails. How large a proportion it would be impossible to estimate statistically, but this cause for running to flange is aggravated by another, which can be rudely estimated.

Trucks Out of Square.

It appears almost self-evident, although your Committee are not aware that it has been demonstrated, that a truck out of square will have a tendency to run to flange even if the axles are parallel. If the axles are not parallel, there will certainly be such a tendency. To fit the axles to run in a 1 degree curve they need only be $\frac{1}{20}$ of an inch out of square. Car trucks, especially for freight, are often rudely made, without much attempt at more minute precision than this, and the statistics gathered by your Committee afford some measure of the resulting tendency to run to flange in the difference in number of sharp flanges in the freight trucks and passenger trucks.

On the Pennsylvania Company lines (Pittsburg, Fort Wayne and Chicago) the percentage of sharp flanges in worn out wheels only is only 11 per cent. in passenger against 20 in freight. On the Chicago, Burlington and Quincy, percentage of all removals, the figures are 6.4 passenger against 21.5 freight. The high percentage given for steel-tired passenger wheels (47.3) may be thrown out of account, for our present comparative purpose, for reasons given in the letter of Godfrey W. Rhodes, Superintendent M. P., subjoined to this report.

Other accompanying data, although not so directly comparable, afford cumulative evidence to the general fact, which is all your Committee can attempt to establish, that with all other conditions approximately equal, a much larger percentage of sharp flanges results in freight trucks than in passenger trucks.

It is probable that from the continued effect of irregular diameters and trucks out of square, a very large proportion of the wheels are subjected to a tendency to run to flange or away from flange; perhaps nearly all of them, and certainly a much larger proportion than finally obtain sharp flanges, so that one or the other of these defects is the first provoking cause of all sharp flanges. That this is so is clearly indicated by another fact of great importance in relation to this question, which

we may now establish, that with very rare exceptions sharp flanges occur only on one axle and are mated with wheels which wear away from flange.

If then other causes combine, such as bad quality, bad chill in the fillet, sharp cornered rails or what not, a sharp flange results ; otherwise not. After establishing the fact just noted we will proceed to estimate the light which statistics shed on each of these latter causes.

Sharp Flanges Occur on One Wheel Only.

So far as your Committee can ascertain, it is practically an invariable rule that only one wheel on an axle has a sharp flange, and by far the most common rule that the opposite wheel is worn away in the tread from flange. Fig. 3, selected by mere chance as the first example of a series of worn sections which came to hand, and made to decide an entirely different question from that now under consideration, illustrates this fact sufficiently well. Out of the sixteen sections shown there are five which show some tendency to wear to flange. In every case the other wheel on the same axle shows an equally marked tendency to wear away from flange.

At the 1883 Master Car Builders' Convention, Mr. Richard Williams, an English mechanical visitor, whose experience had been such as to entitle his opinion to respect, declared as a part of a quite extended discussion, "I defy any of you to say you ever saw two sharp flanges on the same axle," and no one took up the challenge except one member, who suggested that the claim was a little too sweeping. In a paper signed C. E., written by Mr. A. M. Wellington, of your Committee, and published in the *Railroad Gazette* of March 27, 1885, this language was quoted, and it was added : "Modifying Mr. Williams' challenge so as to except wheels which have rendered full mileage before condemnation, the chances are strong that not a single instance can be found anywhere. Certainly, the exceptions are so rare as merely to prove the rule."

This challenge was taken up in a following issue of the *Railroad Gazette* (April 10), by Mr. R. H. Fowle, an Inspector of the Pennsylvania Railroad at the Jersey City shops, in which he stated that he "found that two sharp or worn flanges on the same axle are very common," alleging that he had found two cases in a single lot of 70 wheels, but this assertion was in turn controverted, May 8th, 1885, by Mr. J. N. Barr, Mechanical Engineer (now Superintendent of Motive Power) of the Chicago, Milwaukee and St. Paul Railway, in the following words :

"I was very much surprised at the remark made by a correspondent in the *Railroad Gazette* of April 10th, who says 'two sharp or worn flanges on the same axle are very common,' the same apparently being the result of experience on the Pennsylvania Railroad. The following rule has long been in force on this road and on several others with regard

to using second-hand wheels which are good for further service: 'Wheels showing a tendency to wear at the flange should be mated together; also, wheels showing a tendency to wear away from the flange.' Are not the wheels referred to some that have made a second run and that have been remated in accordance with the above rule?

"As a matter of fact, I would say, after having made this particular point one of careful observation for ten years, that it is an extremely rare occurrence to find two sharp or much worn flanges on the same axle on the first run of the wheels. Two worn flanges on the same axle have been caused by reversing the wheels in the truck after one of the wheels showed considerable flange-wear, but even this often fails to prevent the continual wear of the flange that was worn before the wheels were reversed.

"Any well-conducted wheel record that gives the defects or conditions of each wheel of a pair, will show conclusively that the wear of flanges is confined almost exclusively to one wheel of a pair. From such a record the following figures are given: During the past month forty-five wheels drawn from passenger cars have been reported 'worn flange,' and *in every case* (italics as quoted) the mate wheels are reported either worn tread or worn hollow away from the flange. As to the accuracy of these reports, it should be added that nearly all of these wheels have been twice inspected, first by the men removing the wheels, and secondly by the general inspector for the whole road. The only exception to the double inspection is for wheels that are found good enough to transfer directly to freight cars without being removed from the axles.

"The conditions shown by this months' record repeats itself month after month and year after year, as I have had abundant means of proving.

"Without attempting to theorize in the matter, I present the above merely as a fact; and the question of shape of flanges and rails, which is now being carefully discussed, is of too much importance to be obscured by facts which apparently bear on it, but which, on further investigation, are found to be due to causes which have no bearing whatever on the questions at issue."

Mr. Barr is a gentleman of high standing as a mechanical engineer, and his assertions and explanation would appear to completely neutralize the force of the inspector's statement, which is the only definite claim that two sharp-flange wheels are, in a practical sense, ever found

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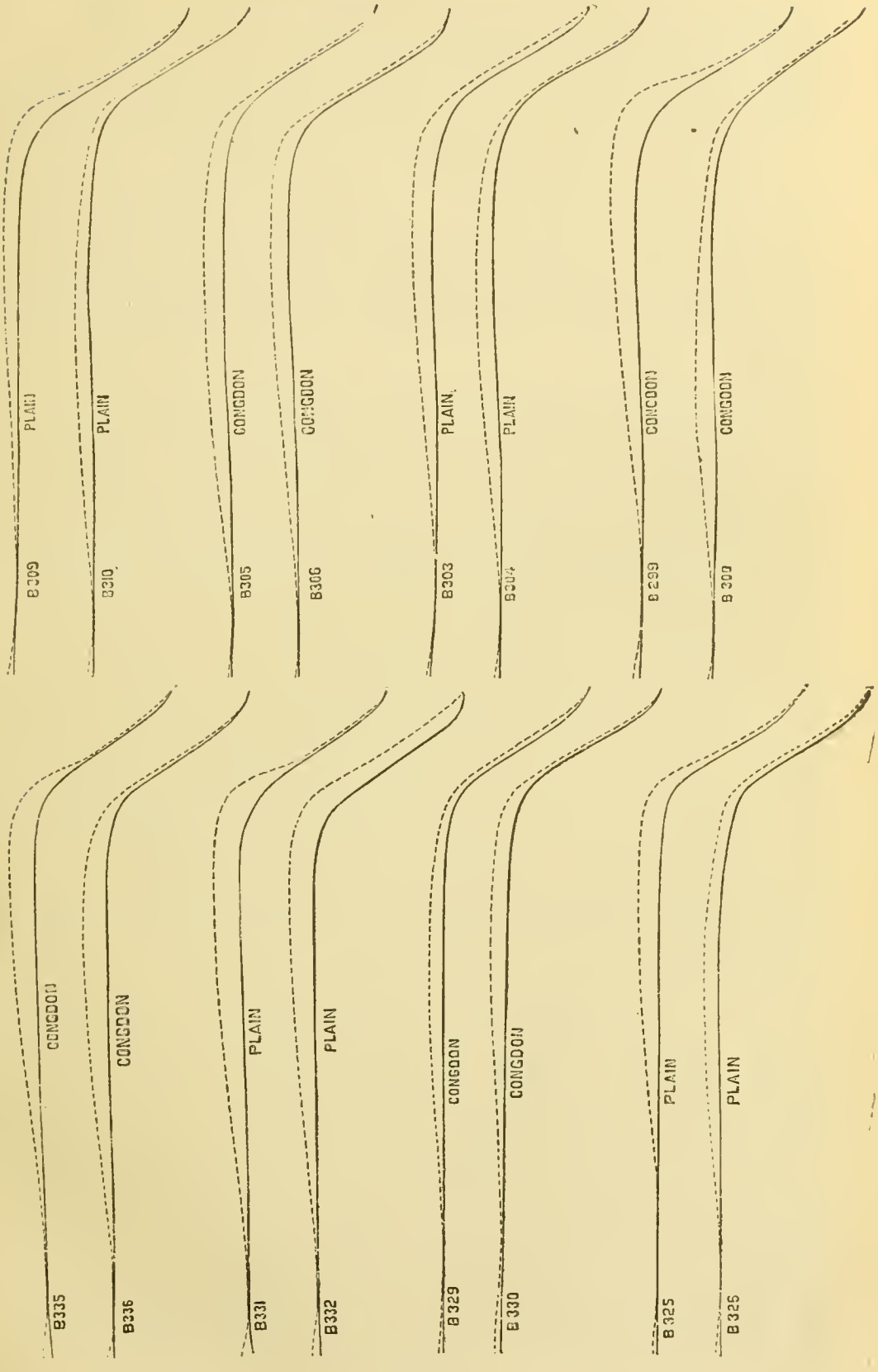
Fig. 3 (on opposite page), illustrates the comparative wear of opposite wheels on the same axle.

It consists of a series of diagrams, reduced to two-thirds of the original size, prepared by Mr. J. Townsend, General Foreman Car Department, Chicago and Alton Railroad, for the Congdon Brake Shoe Company, to illustrate the effect of that device, and not with any reference to flange-wear. He says in regard to the sections:

"The diagram shows the wear of eight steel-tired wheels under Chicago and Alton Baggage Car 40, which was put into service November 11, 1883, also diagram of eight steel-tired wheels under Baggage Car 43, which was put into service November 12, 1883, both cars and wheels being new.

"On the wheels of each car, four Congdon shoes and four common shoes were applied, accurate templates being made at the time, and number of each wheel taken on which the shoes were placed, and is shown in diagram by the plain line.

"At the expiration of six months, the cars having been in constant service between Chicago and St. Louis, accurate templates of the wheels were again made, which are indicated by the dotted lines in the diagram, thus showing the exact wear of the wheels on which were placed the Congdon as well as the common shoes."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B 326
PLAIN

FIG. 3.

on one axle, which your committee have been able to ascertain. The following has further bearing on this question.

On May 29th of the same year a letter was published from Mr. J. D. Hawks, M. Am. Soc. C. E., Chief Engineer of the Michigan Central Railroad, in which he makes the following statements:

"If it can once be settled that the shape of the rail has nothing to do with causing sharp flanges, we can then proportion our rail so as to get the greatest bearing surface for the wheels for each weight of rail. A general master car-builder of a trunk line writes me that 'forty-six 42-inch wheels, with standard tires, under the 60-foot post-office cars running between New York and Chicago, have made an average of 188 123 miles, and not a sharp flange among them. I am pretty certain that sharp flanges are the result of having one wheel on an axle softer than the other.'

"I have noticed many sharp flange wheels taken from under freight cars, where the wheel having sharp flanges shows much wear of the tread close to the flange, while the other wheel on the same axle shows a groove worn in the tread at quite a distance from the flange. This certainly has a tendency to prove that in such cases the sharp flange crowds against the rail on straight lines as well as on curves, and as the wheel without the sharp flange runs on the outside rail of curves as much as the wheel with sharp flange, it seems to show that the flange is worn sharp on straight lines, and that shape of rail-head has nothing to do with causing the sharp flange.

"We have on the Michigan Central one division, the Canada Southern, remarkable for its light percentage of curves. Mr. A. F. Bull, Master Mechanic of that division, says in answer to inquiry:

"I find that in the twenty-six months ending March 1st, 1885, we took out 69 wheels from under engines and tenders on account of sharp flanges. This is 3½ per cent. of the total wheels removed for all causes. Of these wheels, 31 were leading engine truck wheels, 10 trailing engine truck wheels, 17 leading tender truck wheels, and 11 trailing tender truck wheels. But this does not prove that there are more sharp flanges on leading axles, as we leave the trailing wheels on longer."

"That the part (if any) played by curves in causing sharp flanges is very small, is, I think, shown by the above report, as it is not disputed that the trailing wheels do not crowd against the outside rail in rounding curves."

From all the preceding, as well as from personal observation and experience, your Committee see no escape from the conclusion before announced, that, with hardly an exception, there never is but one sharp-flanged wheel on one axle, the other wheel being as a rule worn away from flange. If so, it indicates unmistakably that the primary cause of flange wear is not the rail, but some cause or causes which lead the axle to run more toward one rail than the other. For, since each wheel is subjected to precisely the same rail-wear, if the form of the rail only caused the flange-wear, we should expect to find each wheel equally flange-worn. If it was partly the rail and partly some other cause, tending to make the axle to run to one rail, we should expect to find both wheels somewhat flange-worn, but one wheel much more than the other. Only in the case that the form and area of the normal rail-contact had

no effect to cause flange-wear, and it was wholly caused by a tendency of the axle to run to one rail, should we expect to find one wheel flange-worn and the other worn away from flange. This latter having been shown to be the case, the only possible logical conclusion therefrom is that the form of the rail has no effect whatever in itself to cause sharp flanges, apart from defects of truck or wheel to make the truck run out of true.

Effect of Quality and Make of Wheels on Sharp Flanges.

The evidence which your Committee have collected appears to indicate that the quality of the wheel has a very marked and decided influence on the proportion of sharp flanges, so much so that the proportion of sharp flanges is sometimes from four to twenty or even fifty times as great with one make of wheels as with another, the almost invariable rule being that the poorer wheels have much the larger proportion of sharp flanges. This fact also, if it be admitted to be such, clearly tends to indicate that the form of the rail has very little to do with sharp flanges. For, since both good wheels and bad wheels run over the same rails in the same kind of trucks (in the cars of any one road), the wear due to peculiarities of the rail should be distributed alike between flange and tread. Both flange and tread might be expected to wear faster in a bad wheel than in a good one, but there is no reason why the relative wear of either part should be faster in a bad wheel than a good one. On the other hand, the most difficult part of a wheel to get a good chill on is the fillet of the flange, and it is but natural that badness of a wheel should show there first.

The correctness of this reasoning is supported by experience with wheels "slid flat."

In respect to such wheels your Committee find that, so far as the evidence which they have been able to collect can show, the percentage of failures from that cause is very nearly the same with good wheels and bad wheels, although the absolute number is much greater with bad wheels. It is to be expected that any exterior cause of wear, acting exactly alike on all wheels, like sliding or the form of rail, will have the same proportionate effect.

On the contrary, the ratio of wheels removed for sharp flange to those removed for worn tread or worn out, appears from some evidence of a high character to be enormously greater with bad wheels than good wheels, and your Committee find no evidence to the contrary.

The best evidence obtained as to this effect of quality is an elaborate statement of the wheel removals of 1884, by makers and causes, on the New York, Lake Erie and Western Railroad, given in the Appendix, and abstracted by percentages in Tables Nos. 6 and 7. This statement covers 15 595 removals from 250 973 wheels made by twenty-four different makers, which were numbered by the mechanical department in

TABLE No. 6.

PERCENTAGES OF WHEELS REMOVED in 1884 for various causes of each one of twenty-four different makes, on the New York, Lake Erie and Western Railroad, out of a total of 250 973 wheels and 15 595 removals; wheels by twenty-four different makers.

(Deduced from the Record of Table D, Appendix.)

CLASS I.—SIX BEST MAKERS.—Aggregating 78.2 per cent. of all wheels in service.

MAKERS.	Cracked and Broken.			Shelled out.	Sharp flange.	Slid flat.	Worn flat and worn out.	Total.	Percentage removals to number in service.
	Broken.	Cracked.	Total.						
1	1.2	7.2	8.4	0.3	4.8	9.6	76.9	100.	3.85
2	2.7	6.8	9.5	1.7	1.9	22.9	64.0	100.	3.69
3	0.5	14.9	15.4	0.6	1.6	25.9	56.6	100.	7.40
4	5.3	10.1	15.4	0.7	4.8	29.7	49.4	100.	3 00
5	2.0	23.2	25.2	0.6	2.6	35.0	36.6	100.	2.28
6	3.0	20.8	23.8	0.0	1.2	11.3	63.7	100.	8.97
Average.....	2.0	12.2	14.4	0.7	2.7	22.3	60.1	100.	4.39

CLASS 2.—SIX NEXT BEST MAKERS.—Aggregating 17.2 per cent. of Wheels in Service.

7	2.4	17.0	19.4	0.0	6.9	20.8	52.9	100.	8.21
8	2.5	12.2	14.7	0.1	16.1	20.4	48.7	100.	14.63
9	2.8	30.8	33.6	0.1	6.4	21.0	38.9	100.	9.26
10	1.5	27.9	29.4	0.1	2.9	18.8	48.8	100.	10.88
11	1.3	9.7	11.0	0.3	23.2	33.2	32.3	100.	22.50
*12	3.1	13.8	16.9	0.0	6.1	40.1	36.9	100.	8.23
Average	2.2	23.7	25.9	0.1	8.2	21.1	44.7	100.	10.94

CLASS 3.—TWELVE WORST MAKERS.—Aggregating only 4.6 per cent. of Wheels in Service.

MAKERS.	Cracked and Broken.			Shelled out.	Sharp flange.	Slid flat.	Worn flat and worn out.	Total.	Percentage removals to number in service.
	Broken.	Cracked.	Total.						
13	0.9	71.9	72.8	0.0	0.0	22.3	4.9	100.	14.40
14	3.4	59.3	62.7	0.0	0.0	23.7	13.6	100.	3.76
**15	10.4	19.6	30.0	0.3	14.2	11.9	43.6	100.	90.50
*16	0.0	0.0	0.0	0.0	54.6	45.4	0.0	100.	35.70
***17	6.3	21.9	28.2	0.0	21.8	9.4	40.6	100.	12.62
**18	4.9	14.5	19.4	0.0	24.1	17.8	38.7	100.	19.10
**19	7.9	8.6	16.5	0.0	9.3	14.3	59.9	100.	76.90
*20	1.6	22.9	24.5	0.0	4.9	15.6	55.0	100.	14.10
*21	6.0	36.2	42.2	0.0	4.6	17.4	35.3	100.	28.90
22	0.2	88.8	90.0	0.0	1.6	7.0	1.4	100.	25.20
23	5.8	26.4	32.2	0.0	3.3	49.6	14.9	100.	5.00
***24	7.5	4.5	12.0	0.0	8.9	44.7	34.4	100.	2.29
Average.....	4.4	37.2	41.6	0.0	12.4	20.3	25.7	100.	20.16

Bold face numbers represent makers having from 20 000 to 50 000 wheels each in service. Starred numbers indicate the smaller makers, viz.: *, less than 1 000 in service; **, less than 500 in service; ***, less than 300 in service.

NOTE.—Statements covering the years 1885, 1886 and 1887 have been received since the preparation of this report and show substantially the same facts as the records of 1884.

order of relative merit, according to their best judgment, without a thought of the bearing of the table on this particular question. Because of being thus graded, and because few roads have so large a number of wheels from so many different makers, and with such careful records to abstract from, this record has particular interest, The following is a summary of the percentages deduced from the table in the Appendix, as given in Table No. 6.

TABLE No. 7.

SUMMARY of the percentages of wheels removed in 1884 on the New York, Lake Erie and Western Railroad for various causes.

	Six best makers.	Six next best.	Twelve worst makers.	Average of all on road.
Per cent. of whole number in service	78.2	17.2	4.6	100.0
Broken	2.0	2.2	4.4	2.4
Cracked	12.2	23.7	37.2	19.4
Broken and cracked	14.2	25·9	41.6	21.8
Shelled out	0.7	0.1	0.0	0.4
Sharp flange	2.7	8.2	12.4	5.8
Slid flat	22.3	21.1	20.3	21.7
Worn flat and worn out	60.1	44.7	25.7	50.3
Total removed	100.0	100.0	100.0	100.0

PERCENTAGE OF TOTAL NUMBER IN SERVICE REMOVED FOR EACH CAUSE.

	Six best makers.	Six next best.	Twelve worst.	Total.
Per cent. of whole number in service	78.2	17.2	4.6	100 0
Broken	0.09	0.23	0.88	0.15
Cracked	0 54	2.60	7.50	1.20
Broken and cracked	0.63	2.83	8.38	1.35
Sehlled out	0.03	0.01	0.00	0 02
Sharp flange	0.12	0 89	2.50	0.39
Slid flat	0.98	2.31	4.10	1.35
Worn flat and worn out	2.63	4.90	5.18	3.13
Total removed	4.39	10.94	20.16	6.21

The ratio of sharp flanges to wheels worn flat and worn out is:

Six best makers.....1 to 22.25
Six next best makers.....1 to 5.45
Twelve worst makers.....1 to 2.07

These makers, it should be always remembered, were not classified by number of sharp flanges, but according to their total characteristics, for deciding which the number of sharp flanges was only one of many details, and a very subordinate one.

This becomes apparent by comparing the single makers; No. 1 had 4.8 per cent. of sharp flanges, while No. 7 had only 1.2 per cent., the reason being that No. 1's wheels were a little softer, which gave fewer cracked and broken wheels. Not even the percentage of total removals was used as a basis of classifications, which latter, if examined, will be seen to have been on the whole very judicious, allowing more weight to safety from fracture than any single cause.

Summarizing the indications as to sharp flanges in a little different form :

Out of 1 000 wheels removed there were :

In 1 000 wheels made by the
- Six best makers, 27 sharp flanges.
- Six next best makers, 82 sharp flanges (3 times as many).
- Twelve worst makers, 124 sharp flanges (4.6 times as many).

Out of 10 000 wheels in service there were removed for sharp flanges :

In 10 000 wheels made by the
- Six best makers, 12 wheels.
- Six next best makers, 89 wheels (7.4 times as many).
- Twelve worst makers, 250 wheels (20.8 times as many.)

This leads to the striking conclusion, to the extent that one year's removals from 251 000 wheels by twenty-four different makers can give trustworthy evidence, that the same differences of quality have nearly twice as great an effect upon the number of sharp flanges as upon the number cracked and broken, as thus :

RELATIVE INCREASE IN NUMBER OF WHEELS CRACKED AND BROKEN AND IN NUMBER OF SHARP FLANGES, CAUSED BY DIFFERENCE OF QUALITY.

Comparison by percentages of wheels removed.

	Broken only.	Cracked and broken.	Sharp flanges.
Six best makers, standard of comparison.....	1.00	1.00	1.00
Six next best makers.....	1.10	1.80	3.04
Twelve worst makers.....	2.20	2.80	4.60

Comparison by percentages of wheels in service.

Six best makers, standard of comparison.....	1.00	1.00	1.00
Six next best makers.....	2.56	4.50	7.42
Twelve worst makers.....	9.78	13.28	20.83

[The comparison is made by dividing each percentage of removals given in the summary below the main statement by the percentage of the six best makers standing at the head of the column.]

This evidence is strengthened if taken in connection with the individual record. Thus, "No. 16" had 55 per cent. of his wheels fail for sharp flanges, the remaining 45 per cent. having all "slid flat;" the wheels, it would seem, being actually not good enough either to break, crack or wear. His immediate neighbors above and below, and likewise Nos. 8 and 11, differ from him only in degree, but do differ considerably. All their wheels were too soft. On the other hand, Nos. 10, 13, 14, 20, 21 and 22 have a very small proportion of sharp flanges (for such otherwise bad wheels), and the reason is indicated by the excessive number cracked and broken. Their badness lay in being too hard and brittle.

These conclusions are confirmed by the statistics given in the Chicago, Burlington and Quincy and other records of the Appendix, but in a less clear and decisive way, because of their narrower range. Your Committee therefore has not deemed it necessary to analyse them in detail, but simply refer to them. They find nothing in any of the figures which they have gathered which can be regarded as inconsistent with these conclusions.

Taking all the above figures together, and remembering that the best maker makes some bad wheels, it would almost seem as if differences of quality alone would explain all sharp flanges, without allowance either for trucks out of square, different sizes of wheels or effect of sharp rail corners. This still leaves it possible to suppose that there would be no sharp flanges even with bad wheels if the rail-corners were not sharp, except for the fact next to be considered.

Sharp Flanges a Normal Mode of Failure.

Your Committee deem it appropriate to here point out that a certain

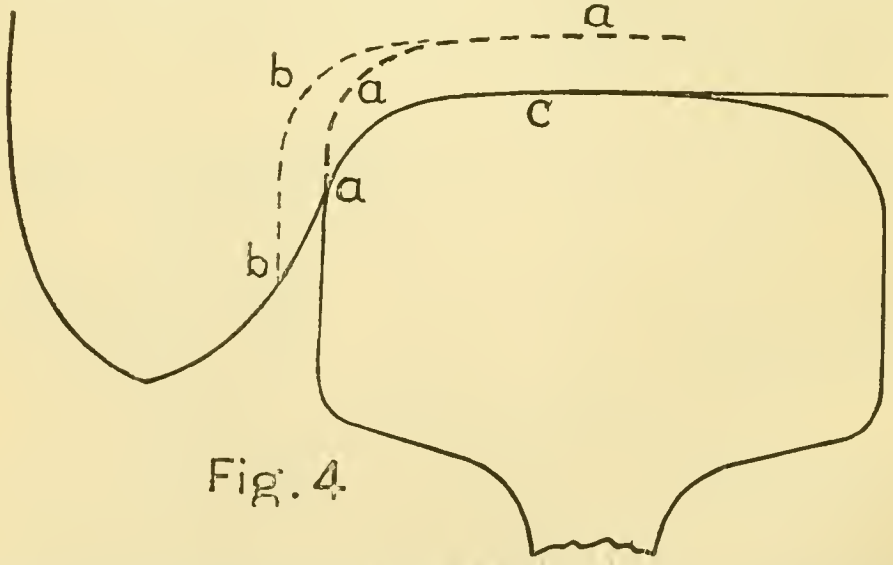


Fig. 4.

proportion of sharp flanges is a perfectly normal and inevitable result from the conditions, and in no case presumptive evidence of any defect

in proportion of either rail or wheel, even without considering the high relative mileage of such wheels before determined, nor does it appear to be a defect in its nature curable by any mode of wear or by any radius of rail corner.

Let us suppose a wheel running only on a perfectly straight road, and running so true that it has no tendency to run to or away from flange. Let us make both fillet and rail corner of large and identical radius, so that no question of the form of rail can come in. Let us suppose that there is no extra abrading friction results from the fillet contact *a c*, Fig. 4, but only normal rolling wear. The result of long service under such impossibly favorable conditions will ultimately be the worn tread *a a a*—which gives a sharp flange. Let there be the slightest tendency to run to flange, and we have the more pronounced case *b b a*. Only in case of tendency to run away from flange will the flange sections remain unmutilated.

Again, suppose the curvature to be so sharp, or the truck so out of square, that before any considerable tread wear results the flange is worn, and let the fillet and corner radii be large and identical as before. We then have the worn tread *c d d*, Fig. 5, and the large corner radius cannot help it.

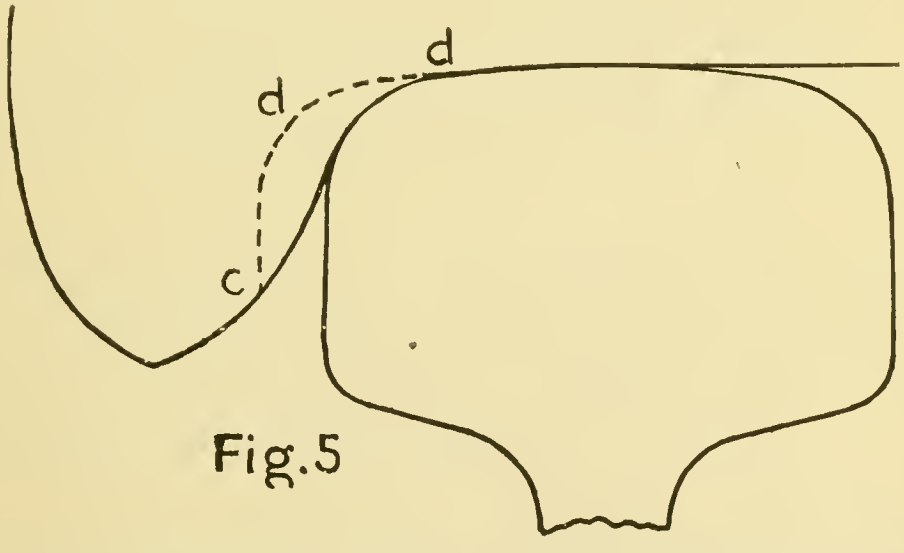


Fig.5

Again, let us suppose as even as possible distribution of the wear between fillet and flange, so that the wear is on a parallel line with the original tread. We still have the straight flange wear *A A*, Fig. 6, and resulting thin flange.

It is therefore no cause for suprise or alarm that when worn out wheels only are considered, excluding wheels which "die young" from defects, the proportion of sharp-flanged wheels is large. If all wheels ran perfectly true the proportion of wheels which showed a decided tendency to sharp flange would be very large, if the above reasoning be correct, approximating to all of them. If we assume, however, as our

reasoning indicates, that practically all axles have more or less tendency to run to flange on one side, this relieves half the wheels of a tendency to flange wear and aggravates it in the other half, and we should expect

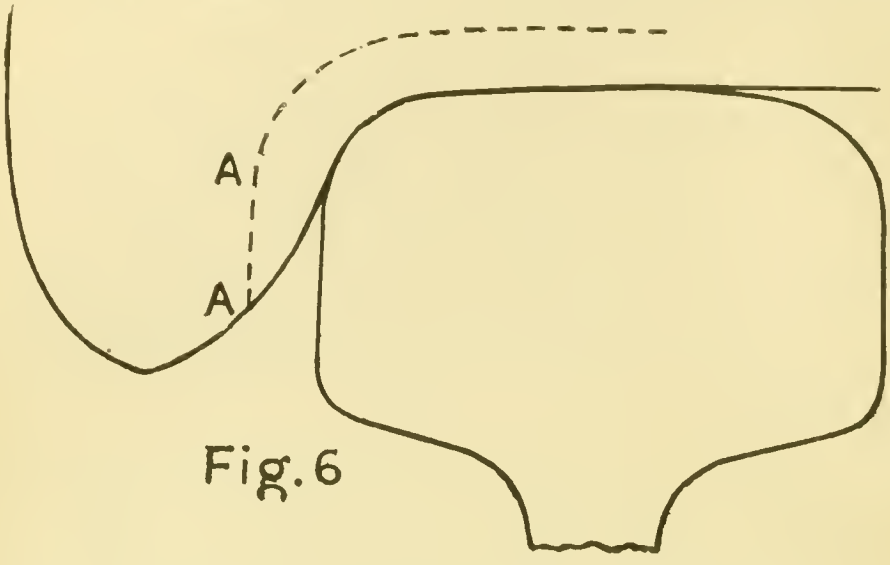


Fig. 6

with more confidence to find a full half the wheels, and no more, suffering from flange wear, whatever the radius of fillet and rail. The statistics herewith presented indicate strongly that when wheels which fail from legitimate wear only are considered, excluding failures from specific defect, this is in fact the case, but the surprising and significant fact is that the life of the flange-worn wheels does not suffer thereby.

Our investigations so far indicate :

First.—That a tendency to sharp flange is a natural phenomenon, which must exist and does exist in all perfect and perfectly running wheels, and is not eliminated by any form of rail or wheel.

Second.—That imperfections of diameter of wheels or of trucks, or both, give a tendency to run to one rail or the other to nearly all axles, relieving half the wheels of flange-wear tendency, and aggravating that of the other half.

Third.—That the quality of the wheel then has an enormous effect on the number of sharp flanges, causing many more when the fillet-chill is softer than the tread.

Fourth.—That when the wheel is sound and the quality even, about half the wheels get sharp flanges, while this proportion is reduced as the number of special defects increases, which cut short the life of the wheel before it is worn out.

Fifth.—That wheels which fail from flange wear are among the most durable of all wheels, if not the most durable, approximating to, if not exceeding, the life of those which fail from long-continued tread wear only, without developing other defect; so that no appreciable money

loss results from wheels running to flange instead of failing from tread wear only.

It remains to be seen, however, what form of rail does most to prolong the life of wheels which run to flange, and what form of flange does most to prolong the life of rails. Considerable statistical light may be thrown on this question from various technical observations of rail-wear on curves and tangents which have been made.

Rail-wear as Modified by Flange Contact.

In the year 1877 Mr. A. M. Wellington, of your Committee, made an investigation of the comparative life of steel rails on the Atlantic and Great Western (New York, Pennsylvania and Ohio) Railroad, the results of which were reported in a paper published in the pamphlet Report of the Road-masters' Meeting of 1878, and also in the *Railroad Gazette* of 1877. These investigations were undertaken for the primary purpose of determining the comparative wear of rails of various makes, and extended to rails of all makes on all kinds of alignment. They were made by determining the original weights of the rail by the carload invoices, removing the rails from the track and weighing them, taking an accurate section by a machine made for the purpose, determining the tonnage which had passed over them, and so determining the loss per yard in pounds per 10 000 000 tons of duty. In all, 145 steel rails were thus computed. The full tabular results are not reproduced here, since they can be easily referred to.

In 1879–80 a similar investigation was made on the Pennsylvania Railroad by Dr. Charles B. Dudley (Transactions Am. Inst. M. E., 1880), 64 rails having been used, 16 from level tangents, 16 from level curves, 16 from grade tangents and 16 from grade curves. Half of the rails from curves were taken from the outside rail and half from the inside rail, and half of each set were selected from the faster wearing rails and half from the slower wearing.

In neither of these papers was the possibility that the rate of wear on curves might vary with the age, and hence, form of the rails given any consideration, or even alluded to. Apparently the possibility did not occur to either of those conducting the investigation. Both papers appear to contain conclusive evidence that such is the case, however, especially when considered together; all the more conclusive because it was unconsciously collected and embodied in tables.

Your Committee know of no other investigations of the kind in this country or in England. Some have been made in Germany, which it has not appeared necessary to refer to further in this report nor to study in detail, since your Committee have reason to believe that they correspond in substance with the facts here presented,

The law of rail-wear on the outside of curves is well known. How-

ever sharp cornered the rail originally, the flange, in time, beds into it deeply, and wears it almost to its exact form, as shown in Fig. 11 below. The newer the rails the less the area of contact surfaces. As the rail wears the contact area becomes greater, until finally it includes nearly the whole flange.

Under these circumstances your committee cannot but regard it as highly significant to find in these records what certainly appears to be absolute evidence that, while the rail is new and the contact areas small, the rate of rail-wear on curves is very slow, but increases in a rapid geometric ratio (approximating to the square of the age in tons, apparently), as the rails become more worn. Since action and reaction are equal, a more rapid rail-wear implies also a more rapid flange-wear, so that if the above fact be admitted it necessarily implies that to imitate in the beginning the conditions which finally obtain from wear on the outside of curves, by making the rail corner and fillet fit each other from *A* to *B* in Fig. 2, is the reverse of good practice.

The following Tables, No. 8 and 9, give in part the evidence which these tests have to shed. There were these differences between the tests:

First—The Pennsylvania tests were made on much older rails, the newest being older in tonnage life than the oldest of the Atlantic and Great Western rails. All the curve rails, without exception, were badly flange-worn.

Second.—The Pennsylvania tonnage had a much less proportion of engines and larger proportion of cars than the Atlantic and Great Western, as did also the tonnage over the Pennsylvania level rails than the tonnage over the grade rails. This greatly affected the rate of tangent wear and curve wear alike, per million tons, making it necessary for comparative purposes to equate the observed curve wear of the Pennsylvania for a uniform tangent wear with the Atlantic and Great Western, on the assumption that the causes, whatever they were, which caused the tangent wear to vary, caused the curve wear to vary correspondingly. The table contains internal evidence that any error in this assumption must have been very small, not affecting the very marked contrast in wear of rails at different ages appreciably.

Third.—The Pennsylvania tests included a much smaller range of curvature, covering only 2, 3, 4 and 5 degree curves.

It will be seen in the following Table No. 9, that without a single exception which can be called such, the effect of age to increase the outside rail-wear is very marked indeed. That it was the change in the form of the rail and not the wearing through of the skin or other cause, which caused this difference of wear, is clearly indicated by the following Table No. 8, showing that the tangent rail-wear was more rapid in the newer than in the older rails.

TABLE No. 8.

TANGENT RAIL-WEAR OBSERVATIONS ON NEW YORK, PENNSYLVANIA AND OHIO (ATLANTIC AND GREAT WESTERN) RAILROAD, 1877.

RAILS OF LEAST AGE.		RAILS OF GREATEST AGE.	
Tonnage.	Wear in Pounds per 10 000 000 tons.	Tonnage.	Wear in Pounds per 10 000 000 tons.
5 526 580	1.24	26 441 000	0.85
6 000 400	1.30	24 788 000	0.85
11 819 800	0.16	21 050 800	1.05
11 259 800	0.66	20 574 800	0.82
14 364 000	1.63	20 574 800	0.44
14 364 000	1.92	20 574 800	1.11
14 364 000	0.63	19 831 000	0.51
14 364 000	0.90	17 433 300	1.09
14 364 000	0.56		
14 364 000	1.84		
Average....	1.084	Average....	0.84

Each of these observations includes the average of 2 to 6 contiguous rails, separately weighted and sectioned to determine loss. Here there is no sign whatever of any tendency to increased wear with age or tonnage duty, but the direct contrary, as was still more markedly the case with the Pennsylvania rails, a fact to be explained largely by the earlier rails being of far better quality than the latter ones. This same difference, presumably, runs through the following statistics in curve rail-wear, and if so, would indicate that the striking increase of curve rail-wear with age which appears, would be considerably greater if the effect of the better quality of the older rails could be eliminated.

TABLE No. 9.

RAIL-WEAR OF THE OUTSIDE RAIL OF VARIOUS CURVES AT VARIOUS TONNAGE AGES, as deduced from tests on the New York, Pennsylvania and Ohio (Atlantic and Great Western) Railroad, showing the increasing rate of wear with age. Corresponding figures from the Pennsylvania tests, equated to correspond to the same rate of tangent wear (0.92 pounds per ton per 10 000 000 tons duty) added for comparison, and to extend the range of the table. (In both sets of tests the tangent rails showed the slowest wear for the greatest ages, or the reverse of the curves).

Curve.	Age of Rails in Tons.	Lbs. per yd. per 10 000 000 Tons.
One degree	1 932 200	1.18
"	14 364 000	0.90
"	14 364 000	0.84
Average, newer rails	10 220 100	0.97
One degree	17 433 300	1.36
"	20 574 800	1.21
"	23 920 000	1.16
Average, older rails	20 642 700	1.24
Two degrees (P. R. R. level average of 4 rails)	57 512 400	1.61
Three degrees	11 259 800	1.09
"	11 259 800	1.69
"	11 819 800	0.34
Average, newer rails	11 446 500	1.04
Three degrees	14 364 000	1.40
"	14 516 100	1.48
"	14 364 000	2 37
"	21 050 800	1.40
Average, older rails	16 073 700	1.66
Three degrees (P. R. R. level average of 2 rails)	46 998 800	2.75
Four degrees	11 950 000	0.81
"	23 136 000	2.42
"	35 910 000	1.74
Average	23 665 300	1.66
Four degrees (P. R. R. grade, slower wearing)	32 428 614	2.23
Four degrees (P. R. R. level, 2 rails)	45 071 000	2.67

Curve.	Age of Rails in Tons.	Lbs. per yd. per 10 000 000 Tons.
Five degrees	14 364 000	3.13
"	14 364 000	3.13
"	21 814 000	1.99
Average	16 847 300	2.43
Five degrees (P. R. R. grade, average of 8 rails)	40 129 300	2 30
Six degrees	6 000 400	1.48
"	11 514 000	1.52
"	14 364 000	2.42
Average	10 626 100	1.81
Eight degrees	5 526 530	2.62
"	5 526 530	2.91
"	5 526 530	3.80
Average, newer rails	5 526 530	3.11
Eight degrees	15 429 300	3.16
"	17 433 300	4.75
Average, older rails	16 431 300	3.95
Twelve degrees	6 000 400	6.92
Sixteen degrees	6 000 400	13.00

NOTE.—The Pennsylvania observed rail-wear, owing either to difference in quality of rails, or to the longer trains and less proportion of engine tonnage, or both (probably chiefly the latter), showed a much lower rate of tangent rail-wear per million tons on the low grades than on the high grades, and even on the high grades than on the Atlantic and Great Western. As the causes which modified the tangent wear would modify the curve wear *pro rata*, the curve wear was equated for the above comparative purposes as follows:

	Tangent Wear per 10 000 000 Tons.		Ratio.
Atlantic and Great Western	0.92	pounds.	1.00
Pennsylvania level grades, average	0.3845	"	$\frac{1}{2.393}$
" high grade, slow wear	0.540	"	$\frac{1}{1.703}$
" " fast wear	0.861	"	$\frac{1}{1.068}$
" " average	0.700	"	$\frac{1}{1.314}$

That these assumptions (as to equating for a uniform tangent wear) lead to correct results is indicated in a measure by comparing the 4 slower wearing and 4 faster wearing grade curve rails of the Pennsylvania tests, the observed actual wear for which was 1.291 pounds and 2.217 pounds. When multiplied by the above ratios they give almost precisely the same equated ratio, as given above.

The inside rails corresponding to the above outside rails show no sign of increase in rate of wear from age in either series of tests.

TABLE No. 10.

RAIL-WEAR OBSERVATIONS on the New York, Pennsylvania and Ohio (Atlantic and Great Western) Railroad, reclassified from Table No. 9, according to Age of Rails when observed.

[The wear is per 10 000 000 tons duty; the duty given is the average of all rails observed.]

	Rails of Least Age.		Rails of Greatest Age.		General Average.	Excess for curve.	Excess per degree.
	Million tons.	Pounds wear.	Million tons.	Pounds wear.			
Tangent.	12.1	1.084	20.0	0.84	1.00	.....	.....
1°.....	10.0	0.97	20.5	1 24	1.10	.10	.100
3°.....	11.4	1.04	16.1	1.66	1.40	.40	.133
4°.....	12.0	0.81	29.5	2.08	1.66	.66	.165
5	14.4	2.65	21.8	1.99	2.43	1.43	.286
6°.....	6.0	1.48	13.0	1.97	1.81	.81	.135
8°.....	5.5	3.11	16.4	3.95	3.45	2.45	.306
Average..	9.9	1.68	19.6	2.15			
12°.....	6 0	6.92	.....	.....	6.92	5.92	.492
16°.....	6.0	13.00	.....	.....	13.00	12.00	.750

	Least Age.		Most Age.	
	Million Tons.	Pounds.	Million Tons.	Pounds.
Average total wear per 10 000 000 tons on all curves 1 to 8 degrees, inclusive.....	9.9	1.68	19.6	2.15
Deduct tangent wear.....	12.1	1.08	20.0	0.84
Excess of wear per 1 000 000 tons due to same curvature at different ages.....	9.9	0.60	19.6	1.31

The concluding lines of Table No. 10 indicate that the wear was as nearly as may be double as fast at double the age.

The conclusions reached above are confirmed independently by another method of analyzing these rail-wear tests. So far we have compared only rails of different age on curves of the same degree. If we compare the average of the rails of different degrees with each other, we are led to substantially the same conclusion by another road.

The slippage per 100 feet of track due to curvature is directly as the degree of curvature, and hence the resistance and the rail-wear should be, other things being equal. If, however, we take a lot of rails of substantially the same tonnage age but on widely different degrees of curvature, and compare them together, it is obvious that the rails on the

sharper curves will have sustained a much larger proportion of their mileage life, have been much more worn to the form of the flange, and hence, if it be a fact that the rate of abrasion increases with such wear, the older rails should show a much more rapid rate of abrasion.

The Pennsylvania tests are too limited in range of curvature, and the rails were all too nearly worn out, to admit of comparison on this basis, but the Atlantic and Great Western were all comparatively new and about the same age (those on the sharpest curves the youngest), and it was found that with observations taken under these conditions the excess of curve rail-wear was as the square of the degree of curvature. In his report of these tests (p. 145) Mr. Wellington stated:

"It is evident from the table that the wear increased considerably faster than the degree of curvature, and it was found that the following empirical formula *fitted the observed facts* (italics as quoted) as nearly as any which could be devised, viz.:

"*Extra wear due to curvature in pounds per yard per* 10 000 000 *tons duty on a d degree curve* $= 0.03d^2$."

And hence,

"*Total wear of rails on a d degree curve in pounds per yard per* 10 000 000 *tons duty* $= 1 + 0.03d^2$.

"In the two following lines is given the wear according to this formula, directly opposite the average observed wear, and the closeness with which the two correspond can be seen.

Observed rate of railwear in pounds, per 10,000,000 tons duty, on a curve of

1°	3°	4°	5°	6°	8°	12°	16°
.10	.40	.66	1.43	.81	2.45	5.92	12.00

Computed rate by formula

.03	.27	.64	.75	1.08	1.92	4.32	7.68

"It is not pretended that this formula is strictly correct, even in theory, but several theoretical considerations indicate that it may be nearly so."

If we remember in connection with these figures, (1) that they included both inside and outside rails, only the latter showing a tendency to flange wear; (2) that the rails on the sharper curves were much younger than on the easier ones, so that the full contrast had not developed, and (3) that the rate of tangent rail-wear of the same rails seemed rather to decrease with age, these figures seem to your Committee to be strong evidence that the rate of rail-wear grows very materially greater as the rail corner is rounded off to fit the fillet of the flange.

Theory indicates that this should be so, because the flange has a longer bearing on the side of the rail, as does also the following additional practical evidence: The old Pennsylvania standard rail was sharp-cornered, with a $\frac{5}{16}$-inch radius; the new one has a $\frac{5}{8}$-inch radius.

These two rails have been examined in parallel tangent tracks by members of your Committee, the old pattern rails being eleven years old and the new pattern only one year old. The old pattern has lost considerably in height, but the rail corner is even sharper than when new, and there is no sign of flange wear. The new pattern is already badly flange-worn for fully $\frac{3}{8}$-inch below the top of the rail, the appearance being that as much metal has been lost off the corner of the rail as on top, and the first impression conveyed being that the track is wide of gauge. It was found on trial that this was not at all the case, however, but that the gauge was full.

Some of your Committee have also observed that the same thing is true of the round-cornered Lehigh Valley rail. Wherever this rail is laid the rail is worn far down the side as if it were the outside rail of a curve, whether on curves or tangents.

The apparent evidence of the experimental records is that the radius of rail-corner which results in least abrasion to the rail, and hence presumably of wheel, is not over $\frac{5}{16}$ or $\frac{3}{8}$-inch, the original corner radius of the A. & G. W. rails tested, and that as the radius becomes larger from wear, and the fillet and flange contact larger, the rate of wear rapidly increases.

It is also a fact so well known that your Committee do not deem it necessary to establish it by further evidence than Figs. 32 and 33 above, that it is only the outside rails of curves which wear to a larger radius to fit the fillet. Whatever the original form, all inside and tangent rails wear in time to a very sharp corner. As an average of the whole United States, curves constitute about 25 per cent. of the whole mileage, and hence outside rails about one-eighth. To establish precisely what does occur, Mr. Thomas Rodd, of your Committee, caused the accompanying sections of worn rails to be taken (Fig. 7), as explained in the Appendix.

Similarly the fillet radius of wheels almost invariably increases with wear, even in flange-worn wheels (see Fig. 3), and so far as your Committee can ascertain, practically never decreases, even in flange-worn wheels. With rare exceptions the fillet radius becomes $\frac{1}{2}$-inch or more.

That the radii of rails and wheels finally become from wear quite divergent does not necessarily imply that they should be made so in the first place, but it does appear to imply that no difference of corner radius can have much effect to diminish flange wear.

So far, your Committee have considered this question purely from the light of experience, with no attempt to determine what theory indicates. It would be improper to draw any conclusions in so important a matter, however, without considering it from a theoretical point of view, which we now proceed to do.

Radii of the Worn Surfaces of Rails in service, Pennsylvania Company (see Fig. 7, next page). Summary of Radii in inches.

TANGENT RAILS—FAST SPEED.

Edgar Thompson steel, 1876, 60 pounds per yard, gravel ballast.

No. of Cut.	Corner Radii. Left.		Right.	
1	....	.265	1.16	.36
2	1.045	.355	0.71	.245
3	0.83	.230	0.82	.28
4	0.82	.23	0.815	.26
Average	0.900	.270	0.876	.286

TANGENT RAILS—SLOW SPEED.

Cambria steel, 1875, 60 pounds per yard, stone ballast.

No. of Cut.		Corner Radii. Left.		Right.	
5, 1875	Cambria	1.15	.20	1.15	.28
6, "	"	0.53	.12	0.67	.215 .008
7, "	"	0.97	.27	0.63	1.65
8, "	"	7.27	.37	0.51	0.85
Average		.844	.240	.740	.211

CURVE RAILS—FAST SPEED.

Pennsylvania West Virginia State line; 7 degrees, 20 feet; 6½ inches elevation; stone ballast, Edgar Thompson steel, 1880; 67 pounds per yard.

	Outside.		Inside.	
9	0.86	0.213	0.24	0.89
10	0.915	0.310	0.25	1.13
11	0.815	0.405	0.30	0.51
12	0.670	0.490	0.31	0.70
Average	0.815	0.355	0.275	0.81

CURVE RAILS—SLOW SPEED.

Six degrees; Wheeling Junction, 5½ inches elevation; stone ballast, Edgar Thompson steel, 1880; 67 pounds per yard.

	Outside.		Inside.	
13	....	.418	0.185	0.555
14	0.630	.485	0.285	0.615
15	1.390	.380	0.155	0.465
16	1.555	.455	0.345	1.045
Average	1.192	.434	0.242	0.670

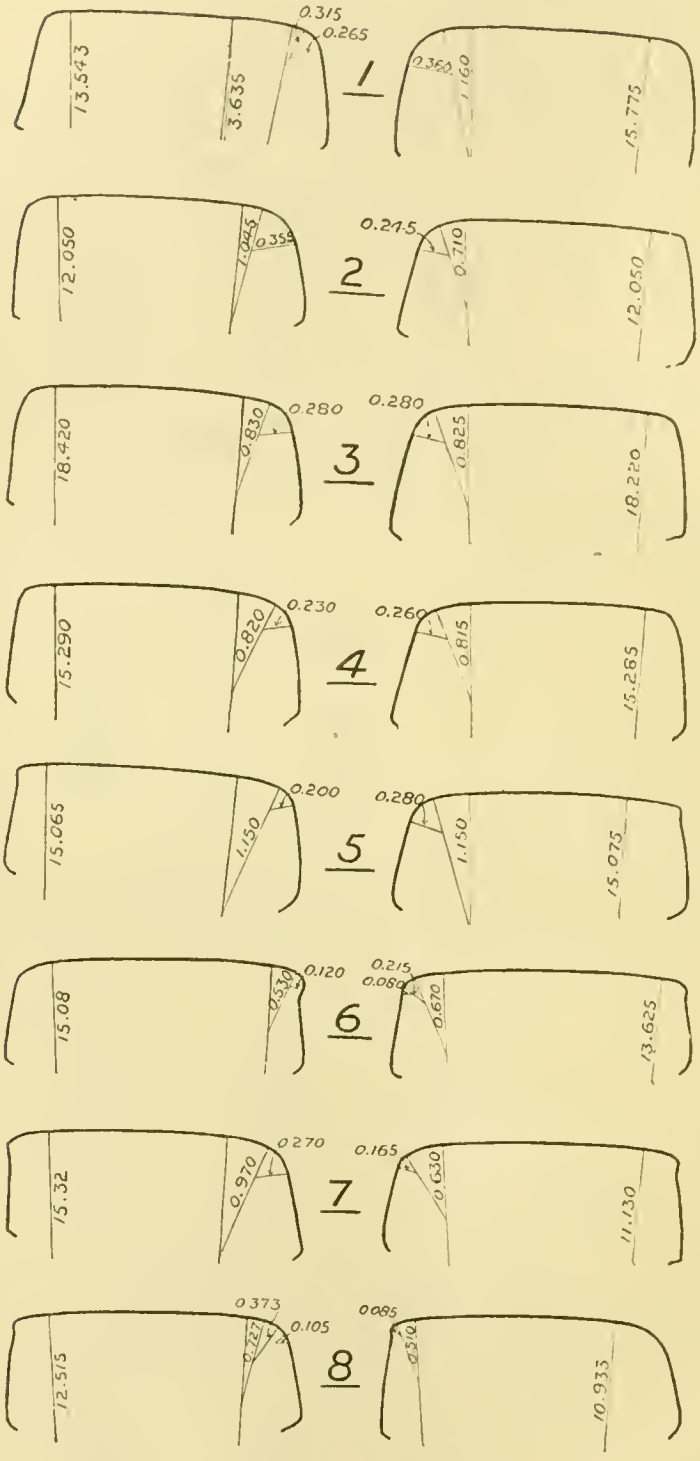


FIG. 7.

Radii of Worn Surface of Rails in Service, Pennsylvania Company.

Tangent Rails.

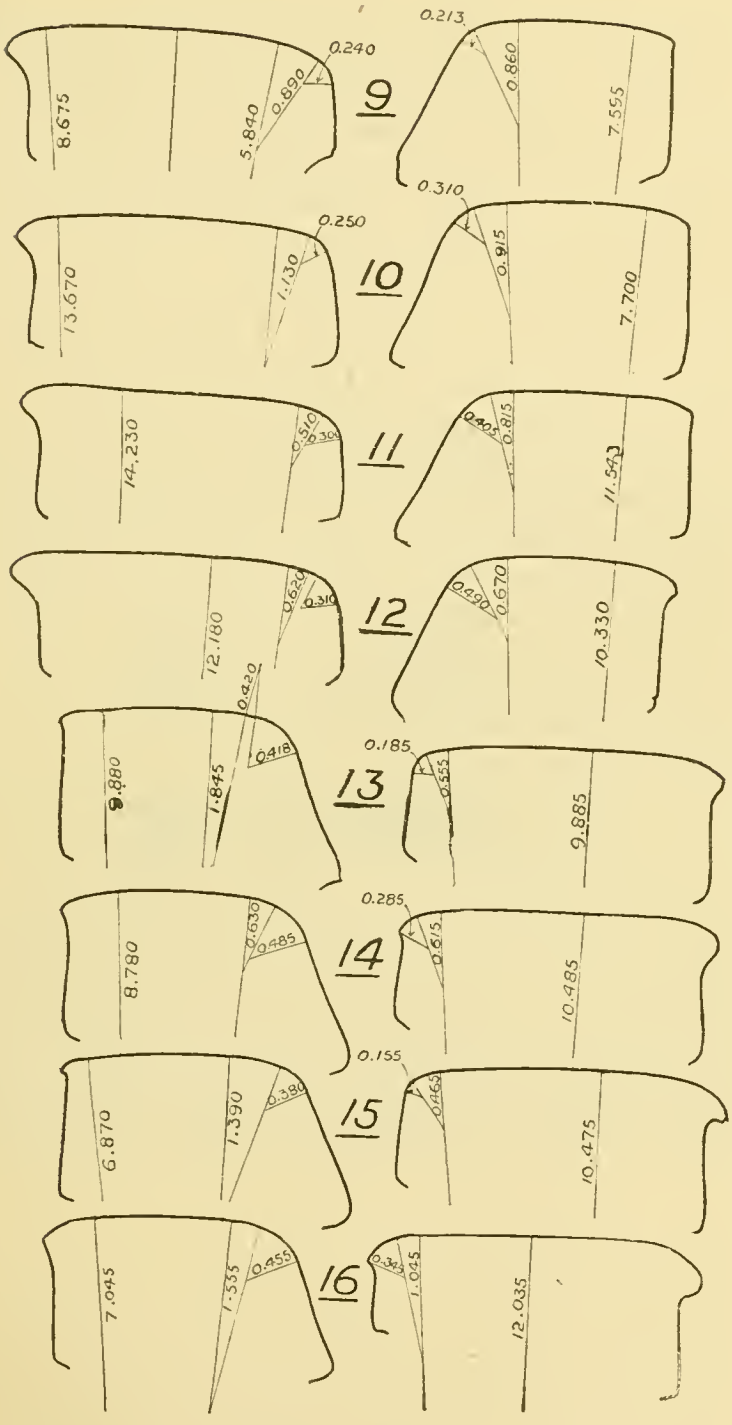


FIG. 7—*Continued.*

Radii of the Worn Surfaces of the Rails in Service, Pennsylvania Company.

Inside and Outside Rails of Curves.

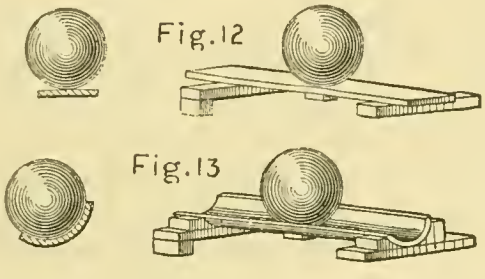
The Theory of the Question.

The wheel has been not unreasonably claimed as the greatest of human inventions, and the essence of the wheel is that its theoretical bearing surface shall be a mathematical line or point, affording no area of bearing surface whatever. In practice this is not strictly the case, owing to the elastic compressibility of the surface, but the bearing surface is always very small, nor can it be increased to advantage by making either wheel or bearing surface more compressible.

To such bearing surfaces the ordinary compression moduli of the text-books have no application whatever. These have been derived from experiments upon prisms, which have the same bearing surface as their greatest section or nearly so, and which fail from flowage or distortion long before the individual molecules of the material are seriously injured. To apply these constants to the bearing surfaces of spheres, or of cylinders laid on their side, or to draw the conclusion that pressures exceeding them are therefore dangerous or destructive, is entirely unwarranted. No attempt has ever been made, nor can there well be, to determine constants for such round bodies per square inch, but on the knife-edges of large scales, which have an angle of some 75 degrees only, and must be and remain practically unworn and uninjured, a load of 7 000 pounds per lineal inch of edge (3½ tons) is considered good practice, or nearly as much as rests on a wheel. If we allow for the difference between this sharp edge, which sustains a continuous load, and the flat tread of a wheel which sustains a load for an instant only, it would warrant an enormous load per square inch of bearing surface. In good bridge practice it is considered entirely safe to load steel rollers rolling on steel up to the equivalent of 10 000 pounds per lineal inch for a 33-inch wheel.

The sphere also has a much less bearing surface than a cylinder of the same diameter, for the same load, yet cannon ball rollers render excellent service in bridge work, nor are your Committee aware that they have ever been claimed to be more destructive to the bearing surfaces.

In figures 12 and 13 we have two opposite conditions for a rolling sphere, and certainly it will be admitted that Fig. 12 shows the best conditions for least friction and wear, although the bearing surface is much less.



Comparative abrasive friction of a sphere, in Fig. 12 with only a point of contact, rolling on a plane, and in Fig. 13 with a bearing surface conforming to the form of the sphere.

Your Committee are not clear that, even if we consider the top bearing only, where no question of consequent rubbing friction comes in, it can be stated without many allowances that increase of bearing surface is in itself an advantage. They cannot regard it as by any means self-evident that less wear would result if each rail and wheel-tread were made a foot wide or even six inches wide. They are rather disposed to believe that it would be entirely disadvantageous.

To see why this is so we must consider what causes rail and wheel wear. It is far from true that, as has sometimes been carelessly assumed, the only wear, or the chief wear, results from crushing, either on curves or tangents. There are these three causes for wear:

First.—Direct sliding of wheel on rail, such as is generally admitted to cause substantially all the extra curve wear.

Second.—Molecular interlocking of the fibres, which is the cause of the wheels revolving instead of sliding, and wears and bends the fibres on their side.

Third.—Direct crushing and disintegration of the fibres from the effect of the insistant weight.

Of these three causes your Committee are decidedly of opinion that the last is by much the least important of all, and they are led to this conclusion by the fact that it seems easy to account for as much wear as occurs by the first two alone. The wear from the first two is most certainly not inversely as the area, but comes nearer to being directly as the area (the total pressure being the same), if brake experiments and others may be trusted. Take direct sliding wear on a tangent, for example. Whenever the wheels are of different diameter direct sliding occurs. It takes a difference in diameter of less than $\frac{3}{100}$ of an inch $\left(\frac{33 \times 56\frac{1}{2}}{5730 \times 12} = .0271 \text{ inches}\right)$ to cause as much slipping as on a 1 degree curve. If the truck is out of square there is constant slipping, its amount depending on the curve in which the truck tends to run. If the flange impinges on either side there is further slipping, both laterally and because of the change in diameter of the wheels. All three of these causes are continually acting, and often to an aggravated degree.

The resulting resistance and abrasion comes from the interlocking of the fibres, and as such interlocking is the only source of propelling power, it must go on for the most part without directly crushing down the fibres, but simply by sliding past them.

In all such friction the wear is certainly very largely increased by having large areas and many fibres in contact. In brake shoes, for example, it is well known that large shoes have greater holding power than small, with the same pressure. So long as the wheel revolves the comparatively small frictional surface below affords as much resistance as the larger brake surface, but the moment the wheel stops, the smaller

surface below causes vastly less friction than the larger one, and your Committee cannot doubt that this is chiefly because there are a less number of fibres exposed to tearing abrasion, and consequently less work done on them. They see no reason why this should not likewise be the case with all the sliding which takes place from causes noted on the top of the rail. In ordinary vehicles, running on ordinary roads, it is not considered advantageous to increase the width of tires more than is necessary to prevent sinking into the surface. While this analogy is an imperfect one, your Committee cannot admit that larger contact of rolling surfaces, and certainly not of rubbing surfaces, results in any decrease of wear, and they see no reason for anxious efforts to accomplish what has never yet been accomplished for any length of time, a bearing surface for the tread extending over the entire top of the rail. This is on the assumption that the rolling contact is of a cylinder of uniform diameter. When we include the corner of the rail and a crowning top, resulting in a different radius of rotation, and hence introduce rubbing friction, the case becomes still clearer, as will next be considered.

In Fig. 2 (reproduced from Fig. 45 of Mr. Forney's paper, above-mentioned, showing the form of rail and flange therein recommended) the tread at B has a radius of $16\frac{1}{2}$ inches and the lower contact point at A a radius of about 17 inches. As wear proceeds the latter radius will become still longer. As the greater load rests at or near to B the wheel will roll on that point, and at all points of contact having a different radius, A or C, will partly roll and partly slide or grind continuously.

A simple computation will show that a difference of $\frac{1}{70}$-inch in the radius at any point $\left(\frac{4.9 \times 16.5}{5730}\right)$ will cause that point to slide as much as the outside (or inside) wheels must on a 1 degree curve. Hence, the point A is sliding on the rail as fast as the outside wheel would on a 35 degree curve, and the whole flange contact from A to B, as fast as the outside wheel would on a 15 to 17 degree curve. This must inevitably produce extra grinding wear, and as there is the same reaction of rail against wheel on curves of all radius (viz., that necessary to continuously slide the wheels laterally so as to change their direction of motion), it will have a tendency to approximate the outside rail-wear on all curves to that on 15 to 17 degree curves.

On the other hand, there can be no question that there is a certain minimum corner radius beyond which the corner cannot be sharpened without destructive wear. To produce an absolutely square section, for example, so that the diagonals of a square section will measure what theory requires, is impossible even in the laboratory. The corners become sharp as a razor, and even the pressure of the finger nail, not to speak of a car wheel, will abrade them so as to materially change the diagonal. A car wheel would produce the same effect with a corner of

quite appreciable radius, and the very least radius which is ever to be found in old rails under any conditions (see Fig. 7) is about $\frac{1}{8}$-inch, nor do your Committee suppose that any one would maintain that rails should be rolled to less than twice that radius.

Between the two extreme limits of (1) an absolutely sharp corner, and (2) the same corner radius as that of the flange fillet, the conditions with new rails are represented by Figs. 8, 9 and 10, Fig. 11 representing the conditions which finally obtain with all rails on the outside of curves. As to how closely it is desirable to have the rail corner under these conditions approximate to the fillet radius the question appears to stand thus:

When the wheel from any cause tends to run against the rail and is restrained by the flange, the forces acting are (1) the vertical component of gravity; (2) the lateral component from the lateral thrust, whatever it is; and (3) the resultant of these three forces, acting at an angle more or less oblique. Under these conditions the laws of mechanics determine what must take place. The wheel will mount until it rolls on the corner with the bearing surfaces normal to the resultant line of pressure, and will then remain in that position. Whatever the respective corner radii, so long as they differ at all, the theoretical line of contact will be a point, and to the extent that compression of the material expands this point into a line, the bearing surfaces will still be normal to the pressure. As the wheel is not revolving in the plane of this resultant, there will still be a certain trifling amount of sliding friction, but the question is:

1. Will there be more or less grinding friction or abrasion under the conditions of Fig. 10 than under those of Fig. 11? It certainly seems to your Committee that there should be greatly less, and the rate of abrasion should increase when the conditions of Fig. 11 are attained and the two radii correspond, as the statistical facts above given seem to show.

2. Is it desirable that the corner radii in Figs. 9 and 10 should be nearly alike, or differ considerably?

So far as rubbing friction only is concerned there will be less wear if they differ considerably; so far as crushing action is concerned, there will be less as they approach each other. As a whole, it seems probable to your Committee that the very minimum rate may be when the fillet and rail radii are nearly alike, but not quite; but that they should be therefore made so in the beginning appears more doubtful for the following reasons:

The evidence seems to be that under all working conditions the rate of rail-wear is very much less before the conditions of Fig. 11 are reached than afterwards. It is therefore desirable to postpone the attainment of such conditions as much as possible. But if the radii be made nearly alike in the beginning we have but little metal to wear away before the conditions of Fig. 11 are attained. In proportion as

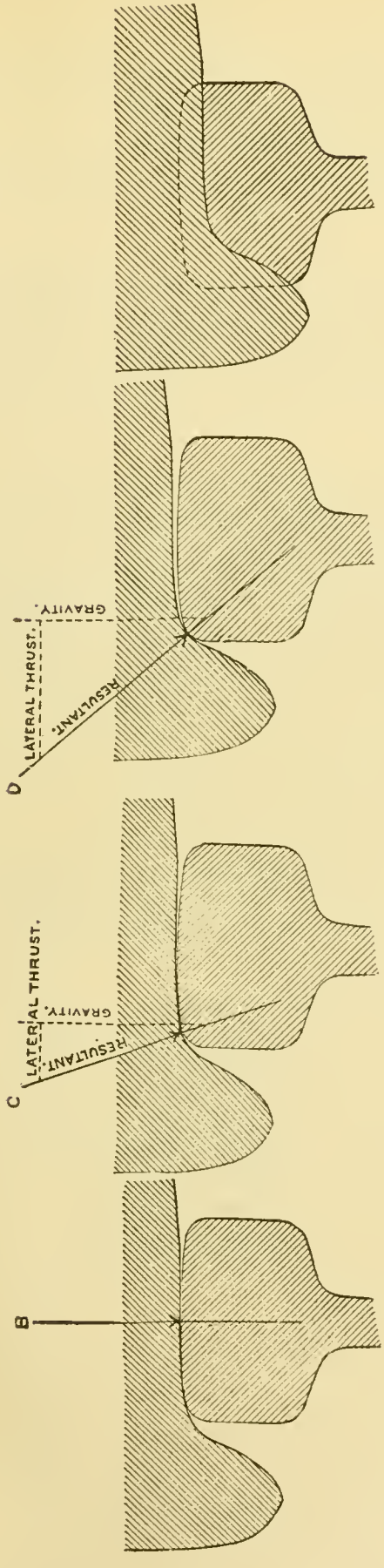


FIG. 8.

Flange acted on by little or no lateral force, but as close to rail as its form permits without such force.

FIG. 9.

Flange acted on by a moderate lateral force, as by impacts or oscillation on tangents. (The line C gives the resultant line of pressure, perpendicular to the bearing surfaces in contact.)

FIG. 10.

Flange impelled against the rail by a strong lateral thrust, as on outside rail of curves. (The line D gives the line of pressure, still perpendicular to bearing surfaces in contact. The wheel rises as shown until they become so.)

FIG. 11.

Worn outside rail on curves, having the bearing surfaces worn to conform to each other, as in Fig 2.

the corner is originally sharper we have more. That is to say, if we take the total metal available for wear in the head at 10 pounds per yard or 1 square inch, by using for the corner radius:

$\frac{1}{4}$	inch in place of	$\frac{5}{8}$	inch we add	14.08	per cent.	to wearing	section.
"	"	$\frac{1}{2}$	"	8.05	"	"	"
"	"	$\frac{3}{8}$	"	3.35	"	"	"

If the preceding indications are correct as to this being very nearly the metal of slowest wear in the wheel head, this is a serious loss, much larger than the above percentages alone indicate, and the question of which radius will give the lowest rate of wear in the new rail is less important than which gives the most metal to be worn away before the conditions of Fig. 11 obtain.

The same reasoning, if sound, applies conversely to the fillet radius of flange. The larger this radius the longer will it take to wear away enough metal to bring about the conditions of Fig. 11. The most usual fillet radius some years ago was $\frac{3}{4}$-inch, but $\frac{5}{8}$-inch has lately been made standard by the Master Car Builders' Association, for the supposed purpose of approximating more nearly to the form of the rail-head, with some admitted disadvantage in manufacturing, because the smaller the radius the more difficult it is to get a perfect chill in the fillet. If the preceding deductions are correct a $\frac{3}{4}$-inch radius is rather to be preferred for all reasons, but there is no particular necessity for having any single standard radius, either for rail or wheel.

The preceding report is submitted for discussion, as before stated, in order that all errors therein may be pointed out, and all available additional evidence brought forward before the Committee attempt to formulate a report giving final conclusions, which they hope to be able to do very briefly.

Respectfully submitted,

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SAMUEL REA,
GEORGE S. MORISON,
THOMAS RODD,
JAMES ARCHBALD,
S. M. FELTON, JR.,
} *Committee.*

TABLE A.

PENNSYLVANIA RAILROAD COMPANY.

Statement showing defects, average mileage, and percentage of WORN-OUT Altoona wheels (wheels cracked or broken, slid flat, etc., excluded) removed from passenger equipment cars during years 1878 to 1887, inclusive.

Baggage, Express and Postal Cars.

Vol. XIX, p. 46.

DEFECTS.	1878.			1879.			1880.			1881			1882.			1883.			1884.			1885.			1886.			1887.		
	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.
Shelled out	115	88 639	23.00	189	64 241	32.36	162	74 647	30.16	117	68 510	14.60	269	63 976	28.60	313	63 190	39.08	221	65 691	25.23	160	76 115	15.69	188	76 726	15.69	191	75 483	16.71
Comby	7	73 199	1.40	17	47 591	2.91	15	53 591	2.79	125	46 891	15.49	106	61 007	11.66	110	64 165	13.73	286	68 222	32 65	397	71 719	38.92	391	72 644	32.64	379	70 369	33.16
Seams	5	88 705	1.00	7	75 798	1.20	5	69 350	0.93	121	31 692	14.99	95	48 695	10.45	79	51 703	9.86	19	68 668	2.17	31	67 855	3.04	18	81 588	1.50	10	88 974	0.87
Worn flat	75	77 132	15.00	79	80 969	13.53	64	86 745	11.92	67	60 693	8.30	159	66 134	17.50	50	72 066	6.24	34	62 531	3.88	18	73 728	1.76	37	77 977	3.09	22	78 368	1.92
Worn flange	146	83 049	29.20	141	90 289	24 14	189	85 508	35.19	250	76 167	30.98	148	74 840	16.28	138	71 695	17.23	208	78 059	23.74	323	73 273	31.67	475	75 367	39.65	454	75 390	39.76
Hollow at flange	43	74 051	8.60	45	81 092	7.71	21	79 781	3.93	16	95 305	1.98	37	70 420	4.07	20	68 535	2.48	18	58 855	2.05	20	79 904	1.96	17	72 054	1.42	14	59 866	1.23
Hollow from flange	104	91 129	20.80	79	95 428	13.53	75	93 318	13.92	42	80 993	5.20	97	71 353	10.67	81	70 802	10.11	88	75 427	10.05	59	82 850	5.78	69	75 008	5.76	72	70 866	6.30
Total																												1 142	73 440	

Combination and Emigrant Cars.

DEFECTS.	1878.			1879.			1880.			1881.			1882.			1883.			1884.			1885.			1886.			1887.		
	No. Drawn.	Average Mileage	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage	Per-centage	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.	No Drawn.	Average Mileage.	Per-centage.	No Drawn.	Average Mileage.	Per-centage.	No. Drawn.	Average Mileage.	Per-centage.
Shelled out	233	65 877	29.29	363	61 287	38.01	803	60 623	45.66	300	59 024	22.54	343	51 989	21.90	408	45 671	32.93	281	44 451	24.31	197	50 493	15.52	241	55 388	11.34	199	58 237	9.59
Comby	14	50 906	1.75	42	46 634	4.49	45	53 307	4 08	188	44 612	14.12	128	54 115	8.21	114	54 356	9.20	312	60 107	26.99	426	58 749	33.57	819	61 741	38.54	810	60 924	39.92
Seams	5	52 819	0.63	7	67 304	0.73	11	52 613	1.00	109	23 200	8.19	143	37 867	9.17	64	44 563	5.17	16	57 333	1.38	27	56 718	2.13	27	56 881	1.27	19	62 916	0.91
Worn flat	132	64 598	16.54	107	69 250	11.20	65	75 955	5.89	94	57 136	7.06	201	57 781	12.88	102	52 237	8.23	70	58 575	6.06	66	56 275	5 20	72	69 965	3.39	54	56 510	2.60
Worn flange	228	72 176	28.57	243	79 846	25.45	307	76 957	27.84	453	62 509	34.03	436	69 323	27.95	281	67 723	22.68	258	60 320	22.32	356	56 701	28 05	715	55 534	33.79	698	60 919	33.69
Hollow at flange	63	71 223	7.89	53	77 318	8.55	49	82 228	4 44	38	76 016	2.85	88	64 587	5.64	37	49 905	2.90	53	51 272	4.58	50	54 768	3.94	52	46 941	2.45	73	37 616	3.52
Hollow from flange	114	82 200	14.29	104	81 376	10.89	94	79 668	8.52	104	70 997	7.81	201	68 371	12.88	176	58 435	14.21	153	55 151	13.24	136	57 287	10.72	190	53 574	8.94	219	50 504	10.55
Total																												2 072	58 624	

TABLE B.

Pennsylvania Railroad Company.

Statement of the total number of wheels drawn; total condemned for all causes; and total worn out, with percentage of worn flange wheels in each class.

Baggage, Express and Postal Cars.

Vol. XIX, p. 46.

	1878.		1879.		1880.		1881.		1882.		1883.		1884.		1885.		1886.		1887.	
		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.
No. worn flanges...	146		141		189		250		148		138		208		323		475	21.06	454	
Total drawn.....	1 208	12.09	1 873	7.53	1 898	9.96	3 152	7.93	2 535	5.84	2 465	5.60	2 418	8.60	2 204	14.66	2 255		2 298	19.76
Less good wheels fit for further service	395		603		556		754		467		472		457		334		395		325	
Total condemned for all causes.....	813	17.96	1 270	11.10	1 342	14.08	2 398	19.43	2 068	7.16	1 993	6.92	1 961	10.61	1 870	17.27	1 860	25.54	1 973	23.01
Less sliding.....	313		686		805		1 591		1 159		1 192		1 085		850		662		831	
Worn out.....	500	29.20	584	24.14	537	35.20	807	30.98	909	16.28	801	17.23	876	23.74	1 020	31.67	1 198	39.65	1 142	39.76

Passenger, Combination and Emigrant Cars.

	1878.		1879.		1880.		1881.		1882.		1883.		1884.		1885.		1886.		1887.	
		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.		Per cent Worn flanges.		Per cent. Worn flanges.		Per cent. Worn flanges.
No. worn flanges...	228		243		307		453		436		281		258		356		718			
Total drawn.....	2 083	10.95	3 104	7.83	3 884	7.90	4 943	9.17	7 379	5.91	7 734	3.63	7 579	3.40	6 701	5.31	6 847	10.49	7 009	9.80
Less good wheels fit for further service	945		1 065		1 433		1 711		2 495		2 029		2 344		1 760		1 435		1 313	
Total condemned for all causes.....	1 138	20.04	2 039	11.92	2 451	12 53	3 232	14.02	4 884	8.93	5 705	4.93	5 235	4.93	4 941	7.20	5 412	13.27	5 696	12.25
Less sliding.....	340		1 084		1 348		1 901		3 324		4 466		4 079		3 672		3 287		3 624	
Worn out.....	798	28.57	955	25.45	1 103	27.84	1 331	34.03	1 560	27.95	1 239	22.68	1 156	22.32	1 269	28.05	2 125	33.79	2 072	33.69

TABLE C.

CHICAGO, BURLINGTON AND QUINCY RAILROAD COMPANY.—COMPARATIVE STATEMENT OF FREIGHT CAR WHEELS REMOVED ON CHICAGO, BURLINGTON AND QUINCY RAILROAD.

Vol. XIX, p 46

Year 1885.

Number of wheels removed for sharp flange.	Number of wheels removed for other causes.	Total number of wheels.	Per cent. of sharp flange to total number.	Make of Wheel.	Average months in use for sharp flange.	Average months in use for other causes.	Life sharp flanges—greater or less. Months.	Per cent.
2 078	7 092	9 170	22.6	Bonton	61	57	4 greater.	7.0 greater
179	296	475	37.7	Griffin	60	54	6 "	11.0 "
33	182	215	15.3	Thatcher	129	156	27 less	20.9 less
85	329	414	20.5	Russell	74	78	4 "	5.4 "
93	373	466	19.9	Chicago	81	69	12 greater.	17.4 greater
18	166	184	9.8	Detroit	70	38	32 "	84.2 "
41	129	170	24.1	Barnum	74	77	3 less	4.0 less
5	311	316	1.5	Griffin & Wells	39	41	2 "	5.1 "
4	27	31	12.8	Barney	46	58	12 "	26.1 "
5	119	124	4 0	St. Louis	73	41	32 greater.	78.0 greater
3	10	13	23 1	Murry	61	106	45 less	73.8 less
2	....	2	....	M. C. R. R	67	....	....	....
2	81	83	2.4	Pullman	60	40	40 greater.	200.0 greater
2	5	7	28.5	B. & O. R. R	87	37	50 "	135 1 "
	69	69		Bass	....	44	.....	.....
	90	90		18 other makes; no sharp flanges	....	....	.....	.....
2 550	9 279	11 829	21.5	Average	63	59	4 greater.	6.8 greater.

Year 1886.

Number of wheels removed for sharp flange.	Number of wheels removed for other causes.	Total number of wheels.	Per cent. of sharp flange to total number.	Make of Wheel.	Average months in use for sharp flange.	Average months in use for other causes.	Life sharp flanges—greater or less. Months.	Per cent.
2 370	8 167	10 537	22.5	Bonton	61	61	....	
203	714	917	22.1	Griffin	63	62	1 greater.	1.6 greater.
21	151	172	12.2	Thatcher	135	165	30 less	22.2 less.
73	317	390	18.7	Russell	83	85	2 "	2 4 "
101	400	501	20.1	Chicago	83	82	1 greater.	1.2 greater.
18	267	285	6.3	Detroit	72	41	31 "	75.6 "
31	184	215	14.4	Barnum	124	92	32 "	34 8 "
11	382	393	28.0	Griffin & Wells	59	50	9 "	18.0 "
1	91	92	4.3	Barney	73	70	3 "	4.3 "
22	121	143	15.3	St. Louis	64	61	3 "	4.9 "
2	14	16	12.5	Murry	92	109	17 less	18.5 less.
2	23	25	8.0	Haskell	147	116	31 greater.	26.7 greater.
4	....	4	....	St. Charles	26	....	.....	.....
1	43	44	2.2	A. C Treat	52	43	9 "	20.9 "
6	74	80	7.5	Bass	89	53	36 "	67.9 "
	229	229		20 other makes; no sharp flanges	....	20	.....	
2 866	11 170	14 036	20.4	Average	64	62	2 greater.	3.2 greater.

CAST-IRON WHEELS REMOVED FROM PASSENGER CARS.

							Months.	Per Cent.
150	2 258	2 408	6.2	Bonton	42.875	26.841	16 034 greater.	59.7 greater.
2	21	23	8.7	R. G. W.	20.328	34 098	13.770 less.	67.7 less.
2	14	16	12.5	Pullman	57 804	53.049	4 755 greater.	8 9 greater.
2	2	4	50.0	Detroit	93 700	64 309	29.391 "	45 7 "
1	....	1		Bass	58.001	.....	.....	.....
	1	1		W. C. W. Co	.....	50.645	.....	.....
	1	1		W. F. Co	.....	17.676	.....	.....
	1	1		Hannibal	.....	2.547	.....	.....
157	2 298	2 455	6.4	Average	43.534	27.099	16 435 greater	60.6 greater.

							Months.	Per Cent.
212	1 848	2 060	10.3	Bonton	36 861	34.555	2 306 greater.	6.6 greater.
6	8	14	57.1	M. C. B	72 201	61 964	10.237 "	16.5 "
6	5	11	54.5	Detroit	67.189	81 631	14 442 less	21.5 less
3	9	12	25.	Pullman	38.412	49.185	10.773 "	28.0 "
	8	8		R. G. W	.....	51.845	.....	
	2	2		Treat	.....	28.556	.....	
227	1 880	2 107	10.8	Average	38.565	34.931	3 634 greater.	10.4 greater.

STEEL TIRED WHEELS REMOVED FROM UNDER PASSENGER CARS.

							Months.	Per Cent.
158	154	312	50.6	Allen	99 563	101 263	1.700 less	1 7 less
8	22	30	26.6	"	52 912	49.194	3.718 greater.	7.5 greater.
12	8	20	60.0	Washburn	42.446	26.257	16.189 "	61 6 "
2	....	2		English spoke	15 891	.....	.....	.....
	16	16		Alston	.....	15 874	.....	.....
180	200	380	47.3	Average 33 inches	43 832	33 615	10.223 greater.	30 41 greater.
				Total average	89.241	85.704	3.537 "	4.1 "

							Months.	Per Cent.
184	146	330	55.8	Allen	85 682	105.265	19 583 less	22.8
10	16	26	38.5	"	86 308	44 187	42.121 greater.	95.3
12	42	54	22.2	Washburn	75.983	70.371	5.612 "	7.9
12	8	20	60.0	English spoke	34.541	23.998	10 543 "	43.9
....	2	2		Alston	.....	30 086	.....	.....
218	214	432	50.4	Average 33 inches	64.333	57.570	6 283 greater.	11.8 greater.
				Total average	82.363	90.109	7.746 less	9.4 less

TABLE D.

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New York, Lake Erie and Western Railroad Company.

Statement showing the whole number of wheels of the several makers, removed, cracked, broken, flat tread, shelled out, sharp flange and flat, by sliding on the New York, Lake Erie and Western Railroad, during the year 1884, with the average number in service per month.

Maker, in order of general excellence.	Cracked plate.	Cracked hub.	Cracked flange.	Cracked treads.	Cracked ribs.	Total cracked.	Broken treads.	Broken flange.	Broken.	Total broken.	Flat treads.	Shelled out.	Sharp flange.	Flat by sliding.	Total.	Average No. in service per month.
1	58	3	6	44	1	112	3	13	2	18	1 201	5	75	150	1 561	40 533
2	60	10	..	38	2	110	31	12	1	44	1 033	28	30	370	1 615	43 873
3	345	5	3	84	5	442	7	7	..	14	1 681	18	47	770	2 972	40 237
4	94	1	1	14	19	129	60	6	2	68	635	9	62	381	1 284	42 844
5	84	6	2	17	5	114	4	6	..	10	180	3	13	172	492	21 546
6	42	8	..	85	3	138	8	12	..	20	420	..	8	75	661	7 376
7	15	..	1	33	..	49	2	5	..	17	152	..	20	60	288	3 514
8	35	4	7	19	47	112	10	12	1	23	448	1	148	187	919	6 274
9	332	18	22	38	43	453	21	19	1	41	572	2	94	309	1 471	15 873
10	395	20	2	43	9	469	15	7	3	25	821	1	48	316	1 680	15 432
11	15	2	1	1	10	29	3	1	..	4	96	1	69	99	298	1 323
12	8	..	..	1	..	9	1	1	..	2	24	..	4	26	65	790
13	58	2	..	6	95	161	1	..	1	2	11	..	....	50	224	1 559
14	33	..	..	..	2	35	2	..	..	2	8	..	....	14	59	1 573
15	18	4	2	46	7	77	35	6	..	41	172	1	56	47	394	436
16	..	..	..	..	..	....	..	..	..	..	..	..	147	122	269	754
17	5	..	..	1	1	7	2	..	..	2	13	..	7	3	32	253
18	6	..	..	1	2	9	3	..	..	3	24	..	15	11	62	324
19	7	4	3	10	..	24	19	3	..	22	168	..	26	40	280	365
20	25	..	..	..	3	28	1	..	1	2	67	..	6	19	122	866
21	68	3	1	6	1	79	9	4	..	13	77	1	10	38	218	758
22	393	1	1	1	..	396	..	1	..	1	6	..	7	31	441	1 752
23	24	8	..	..	..	32	7	..	..	7	18	..	4	60	121	2 426
24	1	..	..	2	..	3	4	1	..	5	23	..	6	30	67	292
Totals	2 121	99	52	490	255	3 017	248	116	12	376	7 850	70	902	3 380	15 595	250 973

Percentages deduced from this table are given in Table 7.

The term "Flat Tread" includes all "worn out" wheels also.

The order of the makers, 1 to 24, was that which, all things included, was considered to represent the general excellence of the wheels, without reference to the number of sharp flanges except as one of many qualities of the wheel.

TABLE E.

Vol. XIX, p 46. WHEELS REMOVED BY THE LEHIGH VALLEY RAILROAD DURING THE YEAR 1886.

Number of wheels removed for sharp flanges.	Number of wheels removed for other causes	Total number of wheels removed.	Per cent. of sharp flanges to total number.	Make of wheel.	Average months in use for sharp flanges.	Average months in use for other causes.	Life sharp flanges greater or less.	
							Months Decreased.	Per cent.
816	8 498	9 314	8.8	Hazleton	113	81	32 greater.	39.5 greater.
489	4 501	4 990	9.8	Catasauqua.....	101	83	18 "	20.5 "
36	855	891	4.0	Cayuta.....	96	57	39 "	68.4 "
22	286	308	7.1	Whitney	205	230	25 less.	12.2 less.
15	157	172	8.7	Lobdell	196	121	75 greater.	62.0 greater.
8	131	139	5.8	High Bridge	181	159	22 "	13.8 "
4	32	36	11.1	Jackson & Woodin	113	158	45 less.	38.9 less.
4	24	28	14.3	J. & N. Scoville.....	125	80	45 greater.	56.2 greater.
....	22	22	0.	Wilmington.....	....	233	..	....
....	12	12	0.	Lambertville.....	....	97	..	....
1	11	12	8.3	Washburn, Hunt & Co....	204	71	133 "	187.3 "
....	9	9	0.	D. L. & W.....	....	133	..	....
8	9	17	47.1	John McDougall	43	42	1 "	2.4 "
15	105	120	12.5	Thirty-nine others.....	72	78	6 less.	7.7 less.
1 418	14 652	16 070	8.8		111	85	26 greater.	30.6 greater.

This table has been abbreviated by condensing thirty-nine makers with one to ten wheels each into one line.

TABLE F.

Pennsylvania Company.

Wheels drawn on account of worn flanges, 1885 and 1886.

33-Inch Passenger Wheels.

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Year drawn.	Worn out. No. of wheels drawn worn out.	Worn out. Average Mileage.	Worn out. Average Service. Yr.	Mo.	Da.	Sharp Flanges. No. wheel worn, flange.	Average Mileage.	Average Service. Yr.	Mo.	Da.	Per-centage worn flange.
1885	843	72 087	....	...	....	92	69 913	....	....	....	11
1886	612	90 833	....	....	....	65	85 705	....	....	....	11
.....	.....	81 460	....	....	....	.....	77 808				

33-Inch Freight Wheels.

1885	4 742	.....	7	5	13	1 066	.....	6	5	4	22
1886	6 233	.....	7	7	21	1 139	.....	6	9	2	18

26-Inch Engine Wheels.

1885	127	90 088	....	....	..	29	87 117	....	....	....	23
1886	82	100 182	.. .	....	....	6	122 886	....	....	....	7

28-Inch Engine Wheels.

1885	203	85 534	....	....	....	27	76 306	....	....	....	13
1886	269	82 835	....	....	....	38	74 556	....	....	....	14

30-Inch Engine Wheels.

1885	501	79 618	....	....	....	109	79 324	...	....	....	22
1886	424	79 914	....	....	....	48	76 948	....	....	....	11

33-Inch Engine Wheels.

1885	767	73 505	....	....	....	182	73 660	....	....	....	24
1886	580	80 190	....	....	....	84	75 573	....	....	....	14
Average all engine wheels..	.....	83 983	....	....	....	.....	83 296				

TABLE G.

BOSTON AND ALBANY RAILROAD COMPANY, 1884.

Wheels Removed from Passenger Service.			Wheels Removed from Freight Service.		
Cause for Removal.	No. Removed.	Percentage.	Cause for Removal.	No. Removed.	Percentage.
Shelled out	65	26.970	Loose	14	2.19
Slid flat	78	32.365	Worn flat	277	43.28
Seamed	2	.829	Slid flat	67	10.47
Worn flat	6	2 489	Cut flange	19	2.97
Honeycombed	7	2 904	Broken	163	25.47
Sand holes	19	7.883	Chipped flanges	7	1.09
Worn out	51	21.161	Worn out	91	14.22
Broken plate	8	3.319	Chipped tread	2	.31
Chipped tread	5	2.075			
Total	241	99 995	Total	640	100.00

APPENDIX

TO REPORT OF COMMITTEE ON PROPER RELATION TO EACH OTHER OF THE SECTIONS OF RAILWAY WHEELS AND RAILS.

Letter from THEODORE N. ELY, Esq., General Superintendent Motive Power Pennsylvania Railroad, transmitting Table A.

ALTOONA, PA., June 15th, 1887.

In reply to your letter in regard to the percentage of wheels removed for sharp flanges, we have gone back over our records since 1878 and give you herewith the information asked for.

These figures show the percentages for the two classes of service which comprise the great majority of our passenger wheels.

The number drawn with sharp flanges from locomotive trucks will probably show a somewhat higher percentage than for passenger cars; this, however, is due to the fact that our inspection is somewhat closer for locomotive truck-wheels than for passenger-car wheels, as their duty is much more severe.

The percentages given are for worn-out wheels only, and do not comprise those removed from sliding, or that were again put in service. You will note that these percentages run from 16.28, the lowest, in 1882, to 39.65, the highest, in 1886. [In 1887, received since, it ran slightly higher.]

There is also sent with these statements, the percentages of average mileage of wheels drawn for the different defects named.

The percentage of average mileage of wheels removed for all causes shows that there is no great difference between those removed from worn tread (or "hollow from flange") and those removed on account of "worn flange."

In the majority of cases, however, those worn "hollow from flange" give the highest mileage, although in some years the reverse was the case. In all cases, however, the average mileage of wheels with these two defects was much higher than for the other causes.

We subdivide the wheels on these statements into two classes, for the reason that the service is quite different. The service of wheels under baggage, express and postal cars, which run principally on through trains with few stops, gives a higher average mileage than the wheels under passenger, combined and emigrant cars, which run principally on local trains, making many stops.

I am free to say that I do not think you will be able to make any deductions from these statements that will enable you to compare them with the percentages of sharp flanges or average mileage of wheels of other roads, for the reason that the service would probably be different, and we feel quite sure from the inquiries that we have made that the inspection of the Pennsylvania Railroad is much more rigid than most other roads, wheels not being worn so closely to their limit.

In this connection I would say that in discussing this matter it should be borne in mind that the shape of the wheel tread and flange cannot so easily be modified as the cross section of the rail, for, in order to get an even chill on the tread and flange, a fillet with a certain radius is found to give the best results, other things being equal.

We keep no record of freight wheels. They are made from same mixture as passenger wheel.

Second letter from Mr. ELY, transmitting Table B.

ALTOONA, PA., June 28th, 1887.

I send herewith a statement from 1878 to 1886, inclusive (Table B, printed in full) which shows the number of wheels drawn for sharp flanges; the total number of wheels drawn; the percentage of all wheels drawn except those which were put into service again, and the percentage for worn-out wheels; this latter percentage is the same as the one before sent you. This statement is in such a shape that you can make any combination of percentages you choose.

I do not think, however, that under any circumstances, the wheels that were good for further service should be used in the comparison of percentages. Many of them were removed because the mate was removed, and not from any fault of the wheel itself, and in the same way, those that were removed from sliding have no bearing upon the percentage for sharp flanges.

Letter of GODFREY W. RHODES, Esq., Superintendent Motive Power, Chicago, Burlington and Quincy Railroad Company, transmitting Table C.

AURORA, Ill., May 4th, 1887.

We send you with this a comparative statement (Table C) showing the number of cast-iron wheels removed from our freight-car equipment during the years 1885 and 1886; also the number of cast-iron and steel-tired wheels removed from our passenger equipment for the same years. The defects, you will observe, are put under two headings, viz.: Number of wheels removed for sharp flange and number of wheels removed for other causes. Under the column Make of Wheel, we show the different kinds of wheels. I do not know, however, as this has any special

bearing on the question under investigation. You will observe that in freight-car equipment the percentage of sharp flange wheels removed to the total number was:

In 1885.....21.5 per cent.
" 1886.....20.4 "

Under passenger equipment we find for cast-iron wheels removed for sharp flanges:

In 1885..... 6.4 per cent.
" 1886.....10.8 "

In passenger equipment with steel-tired wheels this figure is largely increased:

In 1885.....47.3 per cent.
" 1886.....50.4 "

We account for this as owing to the fact that there are many defects in cast-iron wheels, such as shelled out, seams, chipped rim, worn hollow away from flange, etc., which do not occur with steel-tired wheels. Further, the wheels ruined by sliding are very much less with steel-tired wheels than with cast-iron. Many more steel-tired wheels therefore will remain in service constantly subject to flange wear than with cast-iron. Moreover, our steel-tired wheels are used mostly under heavy equipment fitted with 6-wheel trucks. It has been found that the middle wheel of the 6-wheel trucks, notwithstanding the fact that it has no brake shoes to it, wears much more rapidly than either of the other two pair of wheels, and after it has made some thousand miles will almost always be the smallest wheel. I think I am safe in saying it always gives out on account of flange wear and has to be turned down in order to preserve a safe flange and not on account of any wear on the tread.

Mr. W. A. Scott, until recently Assistant Superintendent Motive Power, Chicago and Northwestern, has made some very interesting investigations on this matter, and lately has been running some of his 6-wheel trucks without any flange on the middle pair, and I believe with perfectly satisfactory results. Some modification has to be made in the amount of journal-box play, I believe. If you wrote to Mr. Scott I am sure he would be glad to give you the result of his investigations. He is now Superintendent of the Madison Division at Baraboo, Wis. To show you the proportion of cast-iron wheels ruined by sliding under passenger equipment we give you the following figures for 1886:

Total number of wheels drawn, 2 107.

227 for sharp flanges..... 10.8 per cent.
932 " slid flat..... 44.2 "
948 " other causes..... 45. "

100.

We have, comparatively speaking, but very few steel-tired wheels in passenger service, they are mostly under our Pullman assigned cars, fast mail and heavy baggage cars.

Letter from W. A. Scott, Esq., Superintendent Madison Division Chicago and Northwestern Railway, giving information referred to at close of preceding letter.

Baraboo, Wis., May 17th, 1887.

My investigation developed the fact that with a long wheel base (9 to 10 feet), 6-wheel truck, the center-wheel flanges were cut sharp more rapidly than end wheels, and as steel tires are expensive, we devised a journal box which made the use of center flangeless wheels absolutely safe, and then economized as follows: When end wheels of truck had worn flanges sharp, and they had been turned off until next turning would leave the tire too thin (less than 1½ inches), we turned off flange entirely, thus leaving face of wheel flat, and probably 1¼ or 1⅜ inches thick, and would get a heavy mileage out of this tire located in center; in fact, as much or more than could be obtained from it with flanges, because when flanges were worn sharp it was necessary to reduce tread thickness from ½ to ⅝ inch to secure new flange. It was frequently the case that we could not keep the flanges on a tire and get our 200 000 miles out of same, when by making it flat, when useless for further flange mileage, we could get that much more mileage from same tire, and I think even more, but we have not yet used them long enough to ascertain statistically just how much. My impression is that where 6-wheel trucks are used, one-third maintaining cost can be saved by using a flangeless center wheel, the end wheels (where steel tires ones are used) furnishing enough tire mileage that would otherwise be scraped to keep them going.

Letter from William H. Brown, Esq., Chief Engineer of the Pennsylvania Railroad.

Philadelphia, June 28th, 1887.

I send you herewith some prints of rails lately taken on our road, showing the actual wear from the original section, in which you will notice that the round corner on the rails, which I have been fighting so strongly for the last ten years, is unnecessary, as the corners of the rails actually become sharper by use than when they were first laid.

[The nature of the blue-prints is so clearly explained by the letter that it has not been deemed necessary to reproduce them. They show on rails much worn on top corner radii of ¼ to ⅜ inches.]

Letter from F. D. Adams, Esq., General Master Car Builder of the Boston and Albany Railroad, transmitting Table G.

Allston, Mass., February 14th, 1885.

In answer to your inquiry, I have taken the pains to search our records to reach the information which covers your point.

You will notice that in passenger service no wheel was reported with sharp flange, which indicates, first, that trucks are kept square and in good ordinary condition; and second, that the cast-iron wheel for heavy passenger service will not last long enough, with reasonable care as to exactness in fitting, etc., to wear at the flange sufficient to be condemned for that cause.

The term broken plate could more properly, undoubtedly, be classed as crooked plates instead of broken plates, but this is the term used by the inspectors, and I do not presume to change it.

The report of wheels with cut flanges removed from freight service is interesting as showing, in the first place, in connection with other facts presented, the lighter service, as indicated by the proportion of worn flat and slid flat wheels in comparison with the wheels in passenger service; second, that it is possible that the trucks generally in freight service are not adjusted as perfectly as those in passenger, producing a tendency to run to the flange.

The wheels in freight service represented as broken, may more properly be classed as crooked wheels instead of broken, or at any rate, a large proportion of them, but this is the inspector's definition of the defects.

As to comparative mileage of the wheels removed for the several defects, I am unable to give it; as to wheels in freight service the mileage is difficult to get and is not kept, and the wheels indicated in the report in passenger service had no mileage records kept of their service.

You were correct in believing that the percentage was small of cut flanges.

Letter from F. K. Hain, General Manager Manhattan Railway.

New York, April 18th, 1887.

In response to yours of 11th inst.—During the years 1884–85, 350 wheels were removed on account of bad tread and 6 for sharp flange, a proportion of 1 to 58⅓.

The average mileage for bad tread is 81 514 miles and for sharp flange 62 082 miles.

Letter from L. C. Higgins, Esq., Purchasing Agent of the Lake Shore and Michigan Southern Railway.

Cleveland, O., June 6th, 1887.

In response to the first two questions, viz.: "Proportion of wheels removed for sharp Flanges" and "Comparative mileage of wheels removed for sharp flanges and for other causes," I beg to give you the following figures from our records of passenger, locomotive, truck and tender wheels, removed during the year 1886.

Number of wheels removed for sharp flanges....	397 = 12.2	per cent.
" " " other causes.....	2.864 = 87.8	"
Total removed.....	3.261 100	"

Total mileage of wheels removed for sharp flanges.....	21 268 153
" " " other causes.....	162 060 321
Total.....	183 328 474

Average mileage of sharp flanges.....	53 572
" " other causes.....	56 585

I assume the question does not cover freight wheels as the guarantee given upon them is a period of time, and the fact that they are, a large portion of the time, on foreign roads beyond the jurisdiction of the owner, renders it impracticable to keep any accurate record of the service performed or mileage made before removal.

As to the third question, viz.: "Comparative number of sharp flange wheels among steel-tired and cast wheels," I can make no answer, as we have not yet removed any steel-tired wheels or sharp flanges.

To the remaining question, as to the "effect of quality of wheels on the proportion of sharp flanges," we are unable to give any comparison, as all our wheels are made from a mixture specified by ourselves.

Letter from J. N. Barr, Esq., Superintendent Car Department, Chicago, Milwaukee and St. Paul Railway.

West Milwaukee, Wis., July 4th, 1887.

In my opinion this matter of form of flanges is one of great importance in a pecuniary sense, and one which should be thoroughly and practically investigated. There are so many elements entering into the question, that theoretical investigation is liable to lead to error on account of the difficulty of assigning the proper value to each of the many circumstances that affect the result.

Speaking theoretically, my idea has been that a correspondence between the fillet in the wheel-flange and the head of the rail would be the correct construction if the wheels should slide upon the rail. This is certainly true for sliding pieces; but in the case of rolling pieces it can well be doubted if the reasoning on sliding pieces will apply. In my opinion a close correspondence between the fillet at flange of wheel and the head of the rail would certainly produce a grinding action, which would not only wear the flange and rail unduly, but which would also have an influence on the number of cars which a locomotive could haul.

Looking at the matter in this way, it appears to me that if a form of tread and flange could be designed that would have a tendency to guide the wheel along the track and prevent the flange coming in contact with the rail at all, that improved results might be looked for both as to flange-wear and traction. With this object in view I designed the form of tread and flange shown in the accompanying blue print, dated June 30, 1885. The center of the rail is located on the line marked 10. Toward the outside rim of the wheel the tread, for a distance of $3\frac{3}{16}$ inches, is perfectly cylindrical. Toward the flange the section of the tread increases in diameter for a distance of $\frac{5}{8}$-inch, with a radius of 2 inches. From this point a radius of $1\frac{1}{2}$ inches is used in forming the outline of the tread section, and finally a curve of $\frac{1}{2}$-inch radius extends into the reverse curve of 1-inch radius, which continues to the point of the flange.

It is not possible to say at the present writing whether the proportions given are the best. It may be that the radius used could be still decreased to advantage. It will readily be seen, however, from an inspection of the drawing, that if the rise from the center of the tread to the body of the flange be too much exaggerated, that there will be a tendency to hold the wheel to one point with reference to the rail, and to wear a groove the width of the head of the rail into the tread of the wheel, instead of distributing the wear over the entire surface of the tread. This, of course, would shorten the life of the wheel. Having no data, however, for guidance, it was determined to test the tread in the form shown, and accordingly two chills were made and two wheels were cast in them daily. These wheels were placed in service as made, and a recent review of mileage results is shown in the following statement:

WHEELS WORN OUT.

Defects.	No. of Wheels.	Average Mileage.
Hollow at flange	8	65 471
Hollow from flange	5	83 237
Worn flat	1	60 013
Worn tread	3	78 239
Shelled out	1	12 780
Cracked brackets	4	19 348

WHEELS DRAWN GOOD FOR FURTHER SERVICE.

Defects.	No. of Wheels.	Average Mileage.
Hollow at flange	4	61 738
Hollow from flange	10	57 693
Worn tread	4	72 480

[Worn flanges, none; "hollow at flange" means hollow in tread at flange, not a sharp or worn flange.]

In the matter now under consideration the wheels condemned on account of the defects shelled out and cracked brackets play no part. Investigating the results given above with a view of determining whether the rise from the center of the tread to the body of the flange be too abrupt, it will be observed that there are three wheels condemned on account of worn tread, and that the average mileage, 78 239 is well up to the general average of the worn-out wheels. It may be inferred from this that the rise to the flange is not so great as to confine the wear to a narrow groove in the center of the tread and produce an injurious influence on the mileage.

It should be borne in mind that the failures classified under the head of worn tread, are such that the wear is either at the middle of the tread in a narrow groove or is uniformly distributed over the tread, as distinguished from hollow at flange. The term hollow at flange indicates that the principal amount of wear is close to the flange. The outside of the tread being but little worn, and the flange also being but slightly affected, and the term hollow from flange indicates that the principal amount of wear is on the outside of the tread, that portion of the tread adjacent to the flange being but little worn, and the flange itself being often not worn at all.

The proportion of wheels worn hollow at flange, slight in number, goes to show that there is considerable freedom of play in the wheel across the rail and that the rise or form of fillet at the flange is not so great as to prevent the wheels, under certain circumstances, from breaking and wearing close to the flange. At the same time it should be observed that there is not a single wheel condemned on account of worn flange. I have made it a point to personally inspect these wheels as they were returned from service, and in no case was there a sharp or worn flange, and the general appearance of the tread gave evidence that the flange in sight had but little to do with guiding the wheel on the rail. For purposes of comparison it may be stated that during 1886 ten per cent. of the worn-out wheels arrived from passenger cars were scraped on account of worn flanges, and the average mileage of these wheels was 70 606 miles.

It would not be safe to make any extended deductions from the limited number of wheels with fillet as described, of which record has been obtained, but the results are so encouraging that it has been determined to extend the use of this form of tread and in the course of

about two years we will doubtless have sufficient figures and data to come to a definite conclusion.

I am, however, decidedly of the opinion not only from the mileage figures given, but also from the appearance of these wheels after service, that the best results as to flange wear will be obtained by designing the tread and fillet so as to guide the wheel without bringing the flange into contact with the rail.

With regard to radius of curvature for edge of rail I have not given the matter much attention, and have no data that would throw any light upon this point.

With regard to the effect of quality of wheels on the proportion of sharp flanges, I have not got my data in such shape as to present figures, and it is a matter that could not be very well presented in such a way. The following, however, are my deductions from our wheel records and observations :

First.—It is the exception to find two wheels on the same axle both wheels having worn flanges.

Second.—The conclusion from No. 1 is, that curves have little to do with flange wear.

Third.—The trucks with which I have had experience have little to do with flange wear. This conclusion is arrived at by reversing wheels in the truck when one of the wheels begins to show flange wear, so that the worn flange occupies the position of the good flange and vice versâ. In nearly every case the wheel with the worn flange continues to wear the flange.

Fourth.—If wheels differing in size are mounted on the same axle the small wheel almost invariably wears the flange.

Fifth.—Difference in the wearing quality of two wheels on the same axle is the main, almost exclusive cause of flange wear. The wheel which wears most rapidly being forced against the flange; presupposing of course, fairly good workmanship in trucks and mounting of wheels. If a make of wheels is uniformly poor, there will be less liability to flange wear than if some have very good wearing qualities with the rest poor.

As to the comparative mileage of wheels removed for worn flanges and for other causes, the mileage under passenger equipment of wheels condemned on account of worn flange, is generally a little greater than the average mileage of worn-out wheels (exclusive of sliding) condemned for other defects. This assertion is made from a large amount of data of a definite character which I have not time to present.

As to comparative numbers of sharp flanges among steel-tired and cast wheels it is difficult to make comparisons, as my practice has been to remove and true the wheels up as soon as one give definite signs of wearing the flange, and also to remove such wheels and remate, placing two wheels showing signs of flange wear on the same axle and also mating together those wheels which have preserved their flanges intact. In this way wheels of the same wearing quality are gradually mated together, and the general average mileage increased.

I have now gone briefly over the points referred to in your circular and regret that I have not had time to arrange my figures to re-enforce the positions taken. This is a matter of great general interest, and I would like to obtain copies of what you do in the matter.

PLATE I
TRANS AM SOC CIV ENGRS
VOL XIX Nº 394
KEEFER ON CANADIAN
PACIFIC RAILWAY

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XIX.—August, 1888.)

THE CANADIAN PACIFIC RAILWAY.

ADDRESS AT THE ANNUAL CONVENTION AT MILWAUKEE, WISCONSIN, JUNE 28TH, 1888.

By THOMAS C. KEEFER, President Am. Soc. C. E.

In discharging one of the duties of the office to which I have been elected, I must first express to the Society at large my grateful sense of the high honor conferred upon a Canadian by American engineers —and secondly, my unfeigned regret that engagements (entered into before my nomination) compel me to forego the profit and pleasure of meeting you in Convention, in a year when above all others it would have been both my duty and desire to have done so.

In selecting the subject for my address, I have chosen the "CANADIAN PACIFIC RAILWAY," the most recent, and the most extensive enterprise in a country which, in railway mileage per capita, is second only to the United States. It is not only the most important road in Canada, but it has characteristics which distinguish it from many other railways.

First.—It connects the Atlantic with the Pacific Ocean under one ownership, and upon the shortest American route between Europe and Asia.

Second.—As the single transcontinental line for a country nearly as large as the United States, it possesses the largest area tributary to it of any of the trunk lines.

Third.—No other transcontinental road is so lightly burdened with interest-bearing securities. The whole charge upon 2 500 miles of road is $35 000 000 five per cent. mortgage bonds, $14 000 bonds per mile, or $700 per mile per annum.

Fourth.—As the most recent overland road, it possesses the most modern equipment and the smallest scrap heap.

When the mainland provinces of Eastern Canada (all of which, except the great lake-indented one, Ontario, bordered on Atlantic tide water), became united in 1867 as the Dominion of Canada, the Federal Government took steps to acquire the vast territories between the Rocky Mountains and the watershed of Lake Superior, over which the Hudson's Bay Company exercised jurisdiction—and in which they claimed ownership. Canada refused to recognize the Company as possessing anything more than the right to trade in Rupert's Land, the watershed of Hudson's Bay, and invoked the aid of the Imperial Government. The Company was disposed of by a cash payment of $1 500 000, the retention of their occupied posts, and five per cent. of all lands, lying between the Red River Valley on the east and the Rocky Mountains on the west, and extending as far north as the Great Saskatchewan, which may be surveyed into townships before the year 1920. Over 70 000 000 of acres have already been surveyed, the Company's share of which is 3 500 000 acres, out of which they have sold 470 000. The Company pays for the survey of their lands at the rate of eight cents per acre.

This purchase carried the Dominion of Canada to a line marked by the summit of the Rocky Mountains—between the 49th and the 54th parallel, thence on the 120th meridian to the 60th parallel—which lines form the eastern boundary of the Pacific Province, British Columbia. The 60th parallel is the northern boundary of British Columbia, north of which the Dominion was extended westward to the 141st meridian west of Greenwich, which is the eastern boundary of Alaska in that latitude. This was one of the largest real estate transactions on record.

In 1871, British Columbia entered the Union, thus extending the Dominion of Canada to the Pacific Ocean. The principal condition of this union was that the Dominion should within ten years, connect, by rail,

the seaboard of British Columbia with the railway system of Canada, construction to commence at the Pacific Coast in 1873. Surveys were immediately commenced and prosecuted for about nine years, but the work of construction was not begun until 1875, and then not at the Pacific Coast but at the Lake Superior end. Work was not commenced at the Coast until 1879. Some of the delay is accounted for by the fact that the records of the first three years' surveys were destroyed by fire in Ottawa early in 1874.

The Parliament of Canada had decided in 1872 that the road should be constructed and operated by a private corporation subsidized by the Government, and a contract was arranged in that year with the late Sir Hugh Allan for its construction within ten years, and its operation for a similar period, on the basis of a subsidy of $30 000 000 cash and 50 000 000 acres of land. Sir Hugh controlled a transatlantic steamship line and desired the railway for inland connection. This excited powerful antagonism, and his project was so discredited in the money market that he failed to form his Company. The Government also was defeated on a question arising out of this contract and retired. The new Government was bound to carry out the agreement with British Columbia, but not feeling responsible for its details did not regard time as the essence of the contract, and considered it an impossible one in that respect, especially after the failure of Sir Hugh Allan. It was determined, therefore, in 1874, to proceed with it as a public work, and construction was commenced between Lake Superior and the prairie region in the following year. The Government of 1874 was defeated in 1878, their opponents returning to power, who after continuing the construction as a public work until 1880, reverted to their original policy of construction by a private company. The terms of the contract with the present company were:

First.—$25 000 000 cash and 25 000 000 acres of selected lands in the Fertile Belt, in addition to the right of way for track and stations, shops, docks and wharves, on or through public property.

Second.—Free import of all steel rails and fastenings, fence and bridge material in wood or iron for original construction, and telegraph wire and instruments for first equipment.

Third.—The Government sections under contract—about seven hundred miles—to be completed, with stations and water service, but without rolling stock, and handed over to the Company on the completion

of their contract as a free gift. (The cost of these has exceeded $30 000 000.)

Fourth.—Perpetual exemption from taxation by the Federal Government or by any Province to be created by it, as well as by any municipal corporation of the latter, on all property used for the construction and working of the railway and upon their capital stock. Also similar exemption for the Company's land grant for a period of twenty years, unless sold or occupied.

Fifth.—No line to be chartered south of the railway for a period of twenty years, either by the Dominion or by any Province to be created by it, except for a direction southwest or west of southwest.

The Company contracted to build about 2 000 miles of railway, and to work the transcontinental line for ten years after completion—the standard to be that of the Union Pacific Railway as it was in 1873. Liberal as the terms agreed upon appear, the sequel proved that they were none too much so. The capital stock was fixed at $100 000 000, and it was expected that land sales, or the security of the land grant, would make up any additional amount required and enable the Company to complete the road without mortgaging it. The lands could not be sold while Government lands alongside them were being given away; and land grant bonds, although received at par for Company's lands, could not for the same reason be negotiated to any considerable extent. In the autumn of 1883, $65 000 000 of the capital stock had been sold, and nearly all the proceeds expended in construction. The road was assailed at home and abroad by opponents of the Government and by rival interests, and such distrust created that the remainder of the stock could not be sold to realize the amount necessary to complete the work in hand. The Company then decided to support their stock by purchasing from the Dominion Government a guarantee of three per cent. per annum for ten years, for the $65 000 000 of stock already sold, making similar provision for the $35 000 000 unsold. The cost of this terminable annuity was $16 091 152, calculated at four per cent., to meet twenty semi-annual payments of one and a half per cent. each. Of this amount $8 710 240 was paid in cash and security was given for the early payment of the remainder, but the success of this bold financial policy was defeated by the effect on the money market at this time of the crisis which occurred in the affairs of the Northern Pacific Railway. The stock, with the Government guarantee for so short a term of years,

could not be sold at an adequate price, and the Company, early in 1884, was obliged to apply to the Dominion Government for a loan of $22 500 000. This amount, added to the balance due upon the annuity purchase, made a total loan of $29 880 000, to secure which the Government took a lien upon the entire property of the Company.

In order to obtain feeders and distributors for the transcontinental line, the Company had commenced the construction and acquisition of a railway system in Ontario and Quebec, and branch lines in Manitoba, with a total mileage as great as their contract line, their entire interest in which was transferred to the Government, as well as their unsold stock and their land grant, as security for this loan.

In consideration of this loan, the Company agreed to complete the transcontinental line by May 1st, 1886, five years in advance of the time fixed by the contract.

Railway construction at the rate of nearly five hundred miles per annum rapidly exhausted the loan, and the first lien of the Government over all their property effectually barred the sale of their stock. They found it necessary, therefore, in 1885, to ask that the $35 000 000 of unsold stock in the hands of the Government be cancelled, and an equal amount of five per cent. first mortgage bonds be issued and held by the Government as security for the loan, the mortgage to cover the same security as the loan. The thirty millions loan was payable May 1st, 1891, with four per cent. interest. The Government agreed to accept $20 000 000 of the first mortgage bonds as security for so much of this debt, and the security of the whole unsold lands of the Company (over 20 000 000 acres), for the balance of $9 980 000. Of the $15 000 000 bonds remaining, the Company deposited with the Government $8 000 000 as security for a temporary loan of $5 000 000, and negotiated the remainder, paying back the temporary loan within a few months, and thus releasing $8 000 000 of bonds.

Events took a turn favorable to the Company at last in 1885. The road had been so far completed, that early in that year, and while navigation was closed on the Great Lakes, a military expedition was sent around the north shore of Lake Superior, where there never had been a trail before, to put down rebellion in the Northwest Territories. In November of that year the last spike was driven in British Columbia on a track laid from ocean to ocean. Moreover, the opposition of stock jobbers and rivals subsided, when it was seen that the Government of

Canada were determined to sustain the railway at all hazards. Politically, the existence of the Government depended upon its completion, but the higher consideration was, that the expenditure was so vast and ramified, and the liabilities incurred so great, that suspension would have produced a financial crisis such as Canada had never seen, and one which it was the duty of any Government, if possible, to avert.

In March, 1886, the Company proposed to pay off their indebtedness to the Government, returning all the cash advanced upon the $20 000 000 bonds, and surrendering 6 793 014 acres of land at $1.50 per acre, for the balance. The Government accepted this, and being satisfied that no security was required for continuous operation, surrendered the $5 000 000 land grant bonds held as security for that purpose, but retained $1 000 000 of these to enforce a change of route at Mount Stephen, in the Rockies, where a temporary line of nine miles was adopted to save time and money in opening the road, and upon which there is a grade of 4½ per cent., or double that permitted by the contract.

The year 1887 saw the Company freed from its indebtedness to the Government, and in uncontrolled possession of its property. For the first time since incorporation no legislation for its benefit was required; but in the present year the Government has found it necessary to obtain the surrender of the Company's monopoly as to charters west of Lake Superior, the object of which was to protect their line north of that lake. To effect this, the Government guarantee to pay interest for fifty years on an issue of $15 000 000 three and a half per cent. bonds, secured upon the unsold portion of the Company's land grant—about 15 000 000 acres. The Government becomes a trustee and guarantees the interest for half a century; but not the principal, unless and until placed in funds for that purpose by the Company. The proceeds of land sales are to be funded with the Government, which is to pay three and a half per cent. on any excess of the amount necessary to pay interest upon, or to redeem, the bonds.

The Canadian Pacific Railway is the work of Canada exclusively. The road was undertaken by Canada, as a political and commercial one, to fulfill the compact with British Columbia, and unite together all the Provinces of the Confederacy, but chiefly in order to develop the vast estate purchased from the Hudson's Bay Company. It has been carried out by her people without any assistance from the Im-

perial Government—not even the endorsement of Canadian securities to obtain money at lowest rates, as was done in the case of the loans raised by Canada for the construction of the St. Lawrence canals, and for the Intercolonial Railway. Its importance to the Empire has, however, been recently acknowledged by the British Government, which unites with Canada in subsidizing a line of mail steamers between Vancouver and the British possessions at Hong Kong.

My apology for the space given to the financial history of this railway, is my belief that engineers will wish to know how (as well as why) it has been constructed through a wilderness, and as compared with some of its predecessors, in so short a time. This result is due to the great financial ability, and the still greater courage of its only President, and to the great administrative ability and tireless energy of the Vice-President and General Manager who has had full control of construction and the freest scope in carrying out his plans. Upon the President was imposed the arduous and anxious duty of raising a greater sum than that provided by the Government; upon the Vice-President, the equally arduous duty and responsibility of expending both those sums.

LOCATION.

The general location of the route from ocean navigation at Montreal to the Rocky Mountains, on Canadian territory, is governed by three natural features—Lake Superior, the Lake of the Woods, and Lake Winnipeg. It must go north of the first two and south of the third. Passing from the Ottawa Valley into that of Lake Huron, the line, on its way thence to Lake Superior, traverses for about one hundred miles the watershed of Hudson's Bay near the height of land. In descending thence to Lake Superior, numerous long rock cuts are encountered, separated by shallow valleys, generally with marshy bottoms having little material for road-bed over them or near them except solid rock, boulders and hard pan. Timber of the required dimensions being abundant, the grade was thrown up, shortening the bottom line and reducing the depth of the rock cuts, and trestling was freely resorted to. The embankments have been greatly increased, and much of the rock cutting has been wasted, but it is claimed that from difficulties of position and cost of supplies, the final cost will not exceed that of a slower mode of construction; and, from the local scarcity of bank material, the filling in can be done cheaper by train. The greater

extent of embankment thus obtained has a special value in this district, which has the greatest snow-fall, with the exception of the Selkirk Mountains, of any portion of the route. Along the eastern shore of Lake Superior, high, rocky bluffs are encountered, and heavy rock cuts and numerous tunnels are necessary. The rock excavation runs up to hundreds of thousands of yards on some miles; the cost of one mile is said to approach $700 000.

The highest summit reached between Montreal and Lake Superior is 1 550 feet above tide, or about 950 feet above Lake Superior, and the highest between Lake Superior and Red River is 1 560 feet above tide.

The maximum grade in either direction between Montreal and Lake Superior is one per cent., and the minimum curvature 6 degrees. Between Lake Superior and the Rocky Mountains, the maximum grade going west is, with one exception, one per cent. The exception is a short grade starting from Medicine Hat, a divisional station, where a pusher is always at hand. Coming east, the maximum is 40 feet as far as Winnipeg on the Red River, thence to Lake Superior, 26 feet. This last is the section constructed by the Government.

There is an interesting example of rail creeping on a highly elastic road-bed on this division, where the line crosses a "muskeg"—the Indian term for bog—causing it to yield about 6 inches to every passing train. With a heavy consolidation engine, hauling 35 cars, this track crept 26 inches in the direction in which the train was moving. The rails creep for about three-quarters of a mile east and about half a mile west of a small bridge at the foot of a grade in both directions. They creep with every train, and in warm weather will often run 12 inches under an ordinary train. Track bolts break almost daily, and repairs are to the extent of a box of bolts per month. Cinder ballast keeps the track in line and surface fairly well, but does not in the least prevent the creeping of the rails. Lining and surfacing are necessary at least once a week. On account of the flanges on the angle plates, spikes must be left out of a tie on each side of these plates, otherwise the creeping rail would carry the ties with them, and throw the track out of gauge. Three trains running in the same direction are often sufficient to open all joints on one side and close them on the other side of the bridge between. The whole muskeg, when a train is passing, shows a series of short waves 5 to 6 inches deep, rising and falling with the passing load, and the rails can be seen moving with the passing train.

The General Superintendent of the Western Division, Mr. Whyte, to whom I am indebted for the above, proposes to use 12-foot ties, 40-inch angle bars, and cut a slot in alternate sides of the rail at every tie, as a means of holding the rails in position.

During the construction of this portion of the Railway, two rather serious questions arose out of the interpretation of specifications. The first was upon the work done under the Government, west of Lake Superior; the second upon the work done by the Company east of that Lake. The Government specification for rock read: "All stones and boulders measuring more than 27 cubic feet to be measured as solid rock;" "all large stones and boulders measuring less than 27 cubic feet, and all loose rock, whether in *situ* or otherwise, that may be removed with facility by hand, pick or bar, without the necessity of blasting, to be measured as loose rock." The resident engineers returned as loose rock "large stones and boulders" removed without blasting, but the contractors protested. The question turned on the minimum size of large stones and boulders. Only "large" stones could be counted as loose rock, but there was not this repetition of the word in the case of boulders. The court decided that stones or boulders which were handled and not shoveled were loose rock. Perhaps the words "may be removed with facility by hand," which, as placed and punctuated, apply as well to the stones and boulders as to the ledge rock, influenced the decision.

In the other case, the Company had appointed a successful contractor as Manager of Construction, with plenary powers. The standard Government specification was changed by limiting loose rock to stones and boulders between 1 and 27 cubic feet, and by a special classification for solid rock, the result of the Manager's experience. Some formations of the country rock were known to be harder than others, and no doubt with a view to get a better average, separate prices were taken for granite (the rock of the country), for mica schist and for trap rock. For this last two prices were taken, one for trap in cuttings under, and another in those over 3 feet in depth.

Separate prices were also taken for hard pan and cemented material, but tenderers generally did not recognize any difference in their bids, no doubt because the specification applied the same test for both. It read: "Hard material, where a good picker cannot keep more than two good shovelers going, shall be termed hard pan, or cemented material, as the case may be." The price for hard pan and cemented material

was eighty cents—more than double the earth price; the good picker and the two good shovelers could not be expected to be always together; and however numerous the former might be at any time or place, the "pickings" were good whether the pickers were so or not.

As to solid rock classification, the results on one division were surprising. The geology of the route, which consists largely of metamorphic rocks, was "altered" decidedly. Trap under 3 feet was $3.40 per yard, while granite was $2.20, and more trap was returned at this price than could have existed had all the cuttings been floored with it. When trains could get through, the Company's chief officers found their ballast trains working in cuts where there was no ledge rock, but in which large amounts, including trap, had been returned. They ordered a remeasurement, which was confirmed by another one made by the court; and on one contract the final estimate was reduced between three and four hundred thousand dollars. Other similar cases on this division were settled on the basis of the remeasurement. The section engineers who measured and classified the work in the first instance were generally in accord with the remeasurement, and the sub-contractors were settled with on their classification. After the sub-contractors had been paid off, a revised classification was made out for the final estimate, in which the formation was altered, as effectually as by an igneous eruption, granite merging into mica slate, and trap overflowing everywhere.

The Prairie Section.—The Government had located their line from the Red River to the Rocky Mountains in a northwesterly direction, in order to strike the Yellow Head or Leather Pass, by which route grades of one per cent. were attainable. This direction was also a central one for the largest area of the fertile belt; but it encountered rather formidable crossings of the two Saskatchewans and their tributaries. The Red River crossing is about latitude 50 degrees, and the Pacific terminus but little north of the International Boundary (latitude 49 degrees) while the Yellow Head Pass is in latitude 53 degrees. The route from Yellow Head Pass to the Pacific terminus was necessarily circuitous, and the distance from the Red River to Kamloops* by this pass was 1 350 miles.

The result of the Government surveys showed that the Yellow Head Pass was the most southern practicable one for a line limited to one per cent. grades, and as it was an excellent one in this respect, and was also in the direction of the best line for a colonization road through the prairies, it was adopted. The contract specified this pass, but left the Company their own choice of route to reach it. The question of future competition by parallel lines on the prairies was not considered while

*253 miles from Pacific Coast, and where all lines meet.

it was a Government road. Such lines, so far as they opened up new country, would have been encouraged by a Government within reasonable limits. But with a private corporation the case was different; it could not afford to take the risks which a Government might safely do. The general direction of their road was nearly due west, and as they were on the 50th parallel at the Red River, they decided to abandon the Government location, and to follow that parallel as closely as possible, both to shorten the through distance and to leave no room for a competing trunk line to the south of them. Possibly, also, climatic considerations, in a latitude where every degree counts, supported this deviation. This line traverses the best of the wheat country for nearly 400 miles west of Winnipeg, and the best of stock raising and mining districts, within 200 miles of the mountains. Between these there lies the only section where there is any scarcity of water. Experimental farms have established that there is sufficient rain for crops, and the chief inconvenience to the Company at present is the want of wells for tank supply to the road. Surface reservoirs are proposed to secure these.

In the prairie sections precautions have been taken against snow blockades by keeping the line in embankment wherever possible, and by widening, and flattening the slopes of necessary cuts, and depositing the spoil well off, as a snow screen. At stations, sidings are thrown out so far that cars standing on them cannot cause drifts on the main line. Only twelve miles of snow fencing are used on the prairie section. There was no detention from snow last winter, between the Columbia River and Lake Superior, exceeding four hours at any one time.

There is no good gravel in large quantities for 400 miles west of Winnipeg, but pockets were found sufficient to ballast the wettest portion of the road. The road-bed is crowned off, and top prairie soil is used for surfacing, which gives a smooth-running track for moderately heavy traffic, and one which it is expected will last at least five years.

The Mountain Section.—In Canada the Rocky Mountains maintain a nearly northwest direction, and may be said to terminate as a distinct range between the 51st and 52d parallel; thence descending to the Peace River Pass, latitude 56 north, which is only about 2 000 feet above sea level. All the rivers on the eastern slope of the Rockies penetrate the range to a greater extent the further north they are found, and the Peace River is the first which cuts entirely through the Rocky Mountain range and heads behind it, draining the table land between the coast range and the Rockies. Between Peace River and the International Boundary, some ten passes have been explored, all lowering northward and diminishing from 7 000 to 2 000 feet; the central one, the Yellow Head Pass, with an altitude of 3 733 feet, having been selected by the Government in the first instance as the route for the railway. The

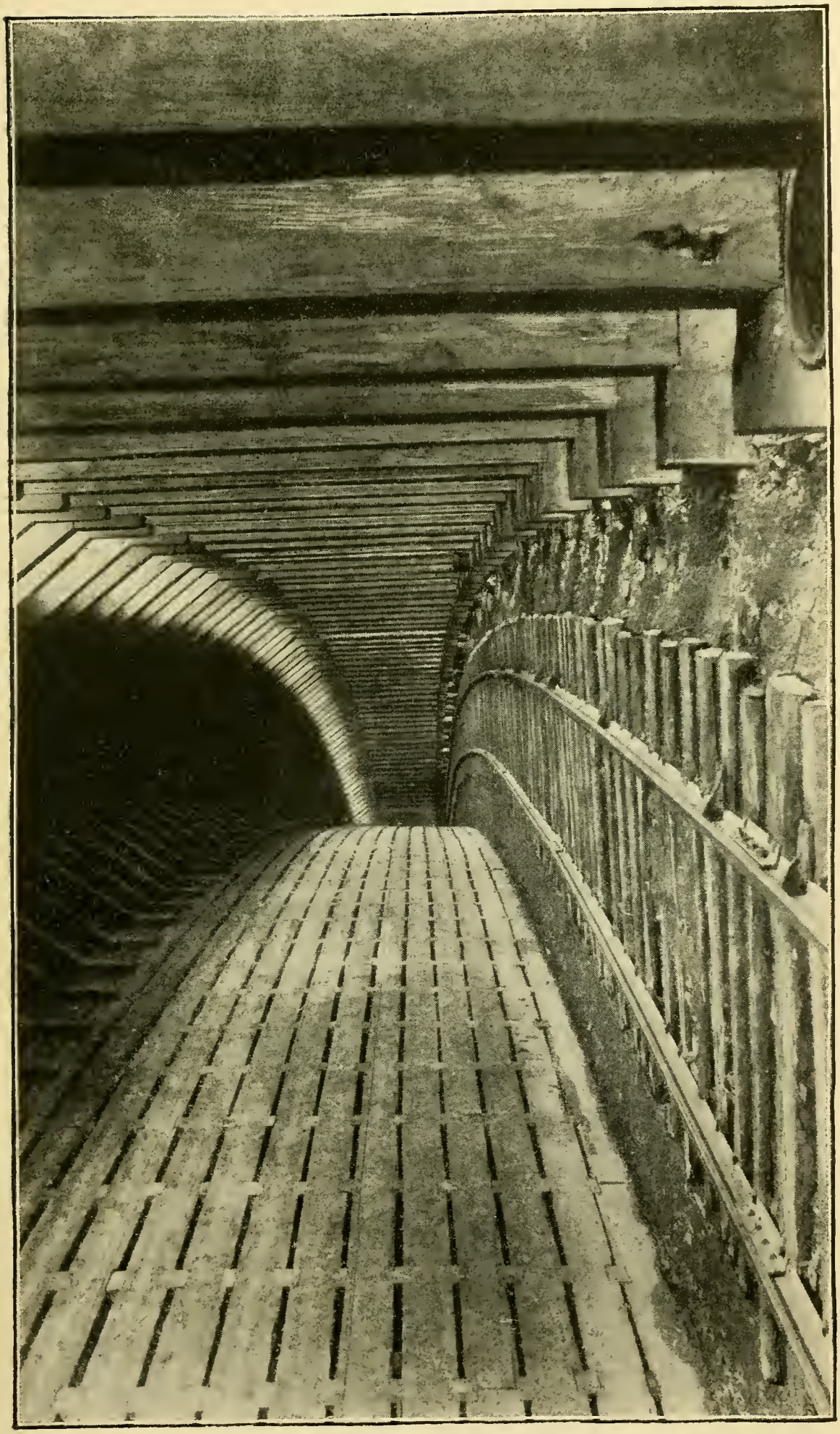
range, which has an average breadth of 60 miles at the 49th parallel, decreases at the Peace River to 40 miles or less.

The "timber line," which in Colorado is about 11 000 feet above sea level, is reduced to 7 000 feet in the Canadian Rockies; and in the latter, above the height of 6 000 feet, snow falls to some extent in every month of the year. Above this elevation, large patches of perennial snow are met with, and it is in the Canadian extension of the Rockies that true glaciers make their first appearance. These, fed by large snow fields, are the sources of the numerous streams which give summer supply to the great rivers of the plains.

The mountain ranges known as the Cordillera Belt, which on the 40th parallel, spread over a longitude of 1 000 miles in Utah, Nevada and California, are here compressed into less than half that width—one of the ranges, an extension of the Olympic Mountains of Washington Territory, being partially submerged in the Pacific Ocean, appearing only in Vancouver and the Queen Charlotte Islands, and reappearing in Alaska. The three mainland ranges are the "Rockies" or Continental Divide (which in Canada shed their waters into the Arctic and Hudson's Bay on the north and east, and into the Pacific on the west), and the "Gold" and "Coast" ranges.

The mountain section extends from the eastern slope of the Rockies to the terminus at the City of Vancouver in the Strait of Georgia, a distance of 522 miles by the railway, but less than 400 as the crow flies —the railway for nearly the whole distance threading its way through the permanent troughs of what has been described as a "sea of mountains."

While driving the line across the plains on their southern location, the Company were seeking a more southern crossing of the mountains, and had obtained the consent of the Government to any pass south of Yellow Head, provided it was at least 100 miles north of the International Boundary. The Government standard of road was one with maximum grades of one per cent., and the Yellow Head Pass lost its superiority when grades of 116 feet per mile were permitted. The Bow River Valley led up by an easy route from the 50th parallel to more than one pass through the Rockies, by which the Valley of the Columbia River could be reached, always with more or less difficulty on the western slope, which is everywhere the most precipitous. In descending the western slope of the Rockies by the Kicking Horse Pass (the most southern available one), the line is transferred from Bow River by ascending one of its tributaries about three miles in length to its source—which is a marsh on the line of the continental divide from which the water at flood time flows in both directions. This marshy summit is 5 300 feet above tide, and the pass is therefore without cutting or tunnel, as well as without a snowshed. Eastward of this point of departure from Bow River there is no grade to Atlantic tide water exceeding one per cent.,



INTERIOR VIEW OF SNOW SHED.

but immediately after crossing the summit, the heaviest grade on the whole line is encountered, four and one-half per cent., upon two stretches of three and one quarter miles each, with a three-mile case between. This is upon what is called "the temporary line," a deviation for nine miles from the contract location (which only permitted half this grade), in order to descend sooner into the bottom of the Valley. The contract location is upon the precipitous face of Mount Stephen, underneath an adjoining glacier, and involved tunneling, increased cost, time and delay in opening the line. There is a temporary curve as well as grade in the Kicking Horse Pass, which was caused by the collapse of a short tunnel from clay expansion. The curve is 23 degrees and is worked by all trains without difficulty at a speed which does not draw attention to the radius. The question will now be between fighting it out on the tunnel line, or avoiding that by a double crossing of the Kicking Horse River.

All the gradients on the Canadian Pacific Railway which exceed one per cent., are concentrated upon the 134 miles which lie between Bow River, three miles east of the summit of the Rockies, and a point one and one-half miles west of Albert Cañon on the Illecillewaet in the western slope of the Selkirks.

The railway route is nearly due west, but the Columbia River, where struck by the Kicking Horse, flows northward to the 52d parallel and there doubles, returning southward and enclosing in this oxbow the formidable Selkirk range of mountains, over which as yet no trail had been discovered. The distance across this range, in the direction aimed at, was less than one third of that following the river around it, and therefore strenuous efforts were made to discover a pass, which, after repeated trials, was effected by Major Albert B. Rogers, M. Am. Soc. C. E., an American engineer. This Selkirk crossing, the summit of which is 4 300 feet above tide, penetrating a previously unexplored region, is one of the few cases in which the locomotive has preceded the Indian in the formation of any kind of trail.

The Selkirk crossing does not show heavy grading for a mountainous section. Its cost is in its structures over and under the track, which are called for by the snow question alone. Strong and costly sheds are required to meet the avalanche, and large and costly bridges only to get out of the way of it.

THE SELKIRKS.

When the last spike was driven in November, 1885, no provision had been made for working the line through the mountains during the following winter. For construction purposes the rails had been laid as rapidly as possible, and the approach of winter suspended completion; but engineers were left behind provided with meteorological instruments, snow-shoes and dog-trains to determine the position, character and ex-

tent of the snow-sheds. The result of this first winter's inspection was the construction in the following summer of 35 snow-sheds, having a total length of four miles. The next winter, the first in which the line was opened for traffic, demonstrated that more sheds were needed, and that existing ones required lengthening in some cases, strengthening in others; that parapets over the portals, and glance-works on the mountain side above were needed to direct sliding snow over the sheds instead of between them. During the summer of 1887, the total length was increased to six miles, and the total number to 53. The experience of the past winter has shown that additions to the sheds are required to the extent of about 4 000 feet, bringing up the total length to about seven miles. The 53 sheds already erected (see Plates V and IX) embrace several types, the primary distinction being, first, those designed for snowfall alone and those exposed to avalanches; and, secondly (as between these last), those exposed to the avalanche on one side only, and those exposed to it upon both sides. These last are called valley sheds, are flat-roofed, and cost about $66 per lineal foot. The typical shed of the Selkirks is an avalanche one, with solid rock-filled crib-work upon the mountain side and strongly braced frame-work for its outer wall. The cost of these range from $40 to $70 per lineal foot, according to location, the increase being due to the greater mass of crib-work required where the avalanche is heaviest. The space between the crib-work and the mountain side is filled in so as to conduct the avalanche over the roof of the snow-shed, without striking heavily against it. The second important type is the gallery shed, which is without crib-work, but has its roof extended against the mountain side upon strong frame-work. The cost of these range from $15 to $40 per lineal foot.

A combination of the typical and gallery sheds is where crib-work is used as a foot wall on the mountain side. This is called "toe crib and gallery" shed, and costs from $27 to $54 per lineal foot.

The gallery sheds are generally extensions of the typical sheds on the flanks of the avalanche and outside its path, and where necessary are terminated by strong parapets as much as 10 feet high to prevent the overflow of the lighter snow from the wings of the avalanche. By means of these parapets 40 to 50 feet of shedding at each end is saved; and the same principle is adopted where slides come down narrow ravines, in which case the profile of the roof is a trough the width of the ravine. These are called "scoop sheds."

During the summer of 1886, fires denuded the mountain sides, leaving no support for the snow on steep side hills, increasing the number of slides and the demand for shed extensions. Sheds were lengthened at each end and connected together until the longest shed exceeded 3 000 feet. Long sheds are objectionable, not only on account of the greater fire risk in summer, but in the handling of long freight trains on the

heavy grades during the winter, when the sheds are entirely dark from snow fall and snow slides. The egress of smoke is then prevented, and brakesmen are unable to see signals or hear whistles. In order to limit the length of sheds, and maintain as many breathing holes as possible, a system of glance-works was devised for the purpose of protecting the necessary openings between sheds. These "split fences," as they are called, are erected on the mountain slope above the track, and are constructed of crib-work or piles, or both. They are triangular in plan, with the apex pointing upward, and on the center line of the snow slides. From the solid triangle which splits the slide, wings are slightly curved and extended, until they pass the line of the shed portals, thus dividing the slide and diverting its course right and left over the sheds. Where there is danger of the snow filling up and overflowing this "split fence," a similar one is placed higher up to cover it. Where only necessary to protect one portal, a glance fence of triangular bents, sheeted with plank, and firmly braced at the back, is planted diagonally with the track, and terminated in strong crib-work at its lower end.

The first winter's experience, founded upon close observation of the character of the slides, proved most valuable in determining the location, design and strength of timber, in the two miles of sheds built the ensuing summer; and by the adoption of wider bents, smaller sized square timber and the more extensive use of the fine round timber, adjacent to the line, for posts and braces, much economy was effected.

The sheds are almost entirely built of cedar, but planking and timbers exposed to transverse strain are of the stronger Douglas fir (Oregon pine) so abundant in the mountains. The cedar in face of heavy cribs is 12 inches square, of lighter "toe cribs" 12 x 10 inches, the back 12 x 12 inches, flatted or round, with 3-inch spaces between the courses. Ties are round, and where the bents are 5 feet centers, break-joints in crib-work every 10 feet, being dove-tailed to the front courses and also to the back flatted timber ones. The saddle-joint is round, and the entire timber work drift bolted together. Dowells are put in foot of plumb posts where gallery is upon toe crib (see No. 4, Plate IX). The joint at the meeting of rafter, plumb, and batter-posts, was, in the first work, so framed as to leave a space for air between the planking of the roof and that on the batter-posts; but it was found unsuitable, because the snow in a slow traveling slide found its way to the track. The joint, as shown in all the drawings, is now used, although not as strong as the first one; the air space is covered by extending the roof, and is kept open until the heavy slides come, when all spaces are securely closed. On this account it is desirable to have the sheds as short as possible, and in view of the success of the split-fence system, suggested by the Vice President and General Manager, Mr. W. C. Van Horne, it is probable that the longer sheds will be cut out at suitable points, and the openings covered by the split fence.

The Telegraph.—Where sheds are in close proximity, an underground cable is used to secure communication with headquarters in any event, and also promptly to locate the site of any interruption. At isolated sheds and suspected points, very high poles carrying the line clear of all probable obstructions, are employed. The only interruption last winter was caused by wind storms, and the loss of time without communication did not exceed four hours.

Fire Protection.—There is a very complete system for fire protection in the Selkirks, stationary and locomotive, gravitation and pumping—stationary for sheds, and locomotive for bridges, buildings, timber, tie and wood piles and forest fires, as well as for the sheds. Water by gravitation is abundant, and flumes are erected on the roofs of isolated sheds, and supplied with running water from the nearest stream, barrels and ladders being placed inside. Where sheds are closer, pipe lines are laid with stop valves at each portal and tanks between, so that damage to pipe in one shed would not affect another. The same system applies to the longer bridges. For smaller ones the usual stationary barrels and buckets are provided.

For the locomotive and pumping system, tanks of 6 000 gallons are kept on flat cars at sidings. Each engine has hose connected with the injector by a globe valve, and can draw from the tender or the portable tanks.

For further protection against forest fires, sand and gravel is dumped from a train around bottom of bridges, trestles, etc.

Where avalanches are expected, the line is thrown well into the mountain side, and the shed roof (which by law must afford a clear headway of 21 feet above the rails) is conformed by back filling as nearly as possible to the mountain slope. Where it cannot be thrown in far enough for this purpose, a broad bench of natural ground is left to take the impact of the avalanche, and send it a tangent to the roof of a comparatively light shed. When the ravines are too deep to be filled up the line is thrown out as far as possible, the ravine bridged with a clear span, the abutments being protected by a glance crib and split fence, and a highway is made for the avalanche to pass under the track.

At the longer sheds an outside or summer track is maintained, both on account of the scenery, which is grandest in the shed region, and also to reduce the risk of fire.

In the 31 954 feet of sheds erected there were used 25 000 000 feet B. M. of sawed material, and 1 140 000 lineal feet of round timber; and the cost of these, including the necessary changes of line to provide for them, of filling in gulleys on the mountain side opposite them, and other work of snow protection, has reached about $2 900 000, and it is proposed to expend about $200 000 more to complete the system.

The sheds were subjected to a very severe test in the winter of

OVERFLOW OF AVALANCHE BEFORE CONSTRUCTION OF GLANCE FENCES.

1886-87, which was more severe than any observed before or since, the snow fall being the heaviest recorded, exceeding 35 feet at the summit in the Selkirks. Eight and-a-half feet fell in six days, and for about three weeks snow was falling almost continually, and slides during this period were very numerous and constant. The sheds proved strong enough in every respect, although subjected to the weight of snow 50 feet deep, weighing 30 pounds to the cubic foot.

The warm Chinook winds and winter rains, followed by frost 30 degrees below zero at times, make the snow very heavy. It has been repeatedly weighed and varies from 25 to 45 pounds per cubic foot, the latter kind being compacted in masses of 5 to 15 cubic yards, and looking more like ice floes than snow balls.

Before the snow-sheds were erected, side cuttings on the slopes exposed to slides were obliterated by the latter, and the *statu quo* of the original snow slope was restored. The snow, which generally brought rocks and trees with it, was packed by the great pressure of the slide so as to be nearly as hard as ice. Black powder was found to work admirably in the side-hill cuts—huge masses being blown down the hill—and the remainder, in heavy blocks, was rolled over the side. In thorough cuts powder was used in heavy charges to break up the snow which was too hard for shovelling, except near the top. Picks and specially-designed ice chisels were here used, and the cut was benched out, entailing a large amount of labor in casting.

The following sketch shows the manner in which the powder was applied in side-hill cuts:

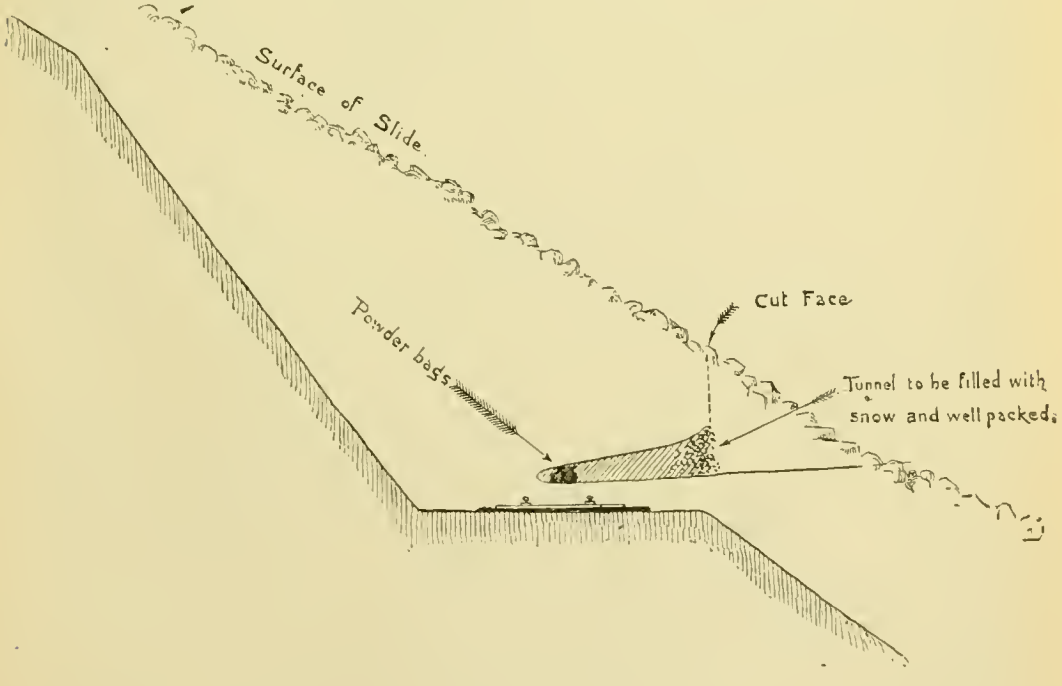


Fig. 1.

One cutting, about 40 feet deep, was full of trees, and presented such a peculiar appearance after being gulletted for the passage of trains, that it received the name of the "Plum Pudding."

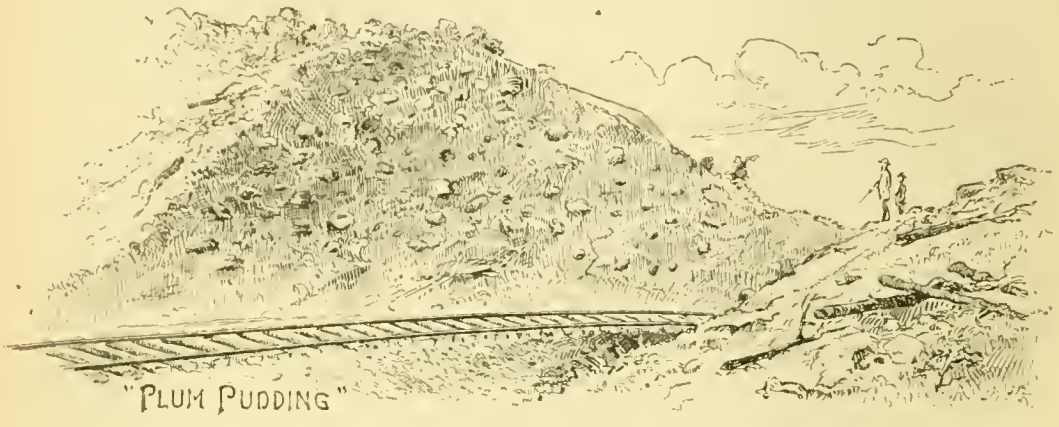


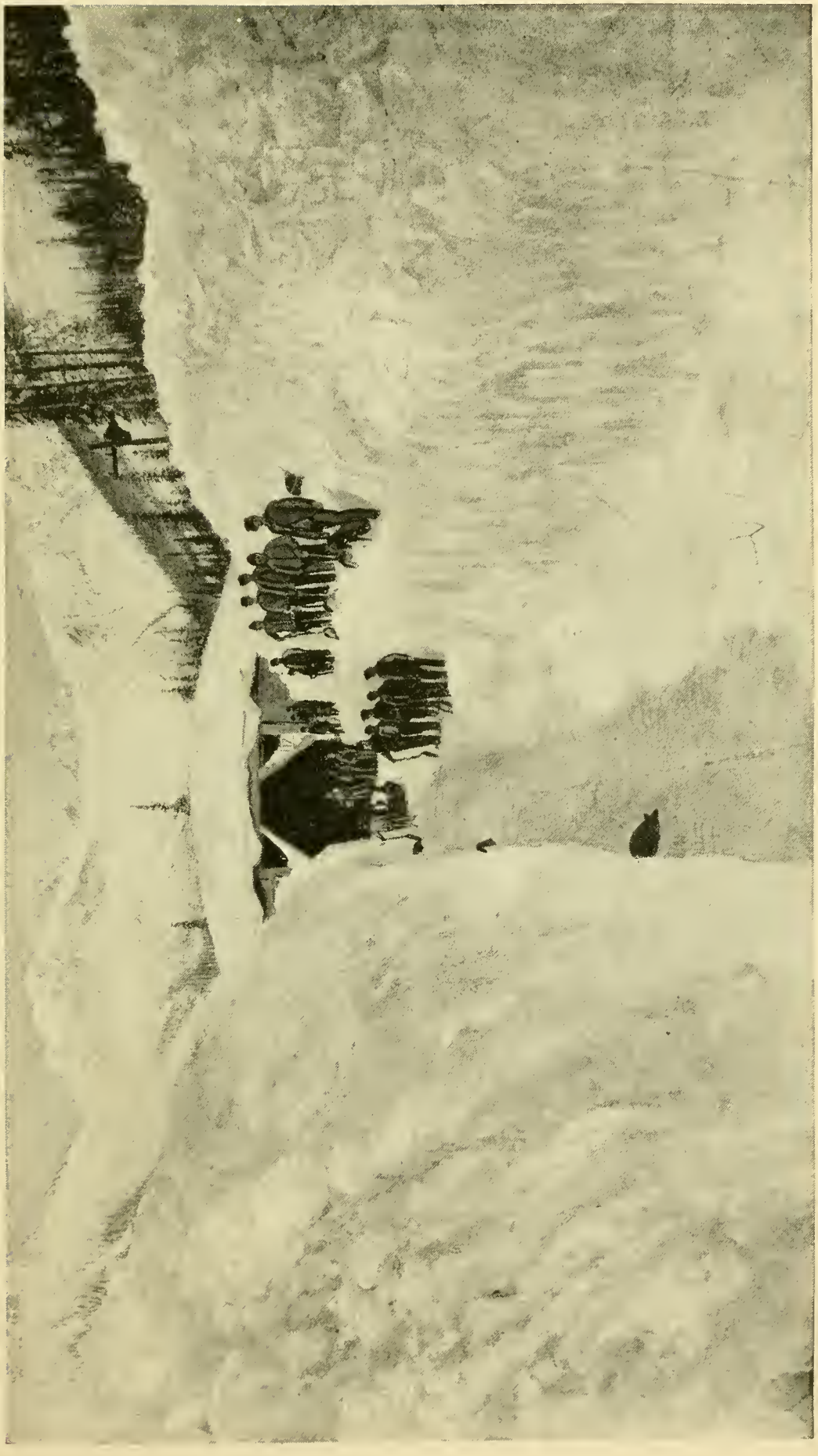
Fig. 2.

The force of some slides was shown by the experience of a valley or double crib shed exposed to them from both sides, the unfinished lower side of which (see No. 2, Plate IX) was left without the batter-post and sloping sheet of plank. When struck on this side, the crib, though filled with stone, was knocked a foot out of plumb, causing the rafter to buckle, the roof being torn off and carried 200 feet up the steep slope above the track. The inside of shed was filled up with snow, which was piled 30 feet deep above it. When clearing out the shed, spaces large enough for a man to go through were found at several points, evidently the effect of confined air, and indicating the rapidity of movement which prevented its escape. Further evidence of air compression was found in the spaces between the wall timbers, which were caulked with snow so hard that no impression could be made upon it without using a pick. It is a question in this case, whether the roof was torn off only, or partly blown off by air concussion.

The sheds exposed to the descent of heavy trees, ice or rocks, have the roof double planked (see No. 6, Plate IX) with intermediate rafters and posts. A rock slide of 100 cubic yards passed over one of these, leaving a specimen rock measuring 128 cubic feet, about ten tons, on top of the roof. In this case the slope of the ground above coincided very closely with that of the roof, exposing the latter only to a rolling load.

The snow-slides vary in intensity from the quiet descent on the slope to the rushing avalanche, bearing rocks and trees, and accompanied, as it always is, with a terrific cyclone more dangerous than either. They sometimes bring down a quarter of a million cubic yards, and are governed by the moist or dry condition of the snow, by the varying slopes of the mountains, the presence or absence of trees, and of sloping crests many thousand feet above grade in the region of eternal snow and of maximum precipitation.

The avalanche is "made up" by excessive snow-fall 4 000 to 5 000



SNOW CUT, SUMMIT OF SELKIRKS, BEFORE EXTENSION OF SHED.

feet above the level of the track, and "pulls out" over the sloping surface of a glacier, or of old packed snow of the previous winter, until it reaches the steep grade (in some cases at an angle of 70 degrees) and then there is a roar, a crash, a flying scud of snow, and all is over. Its maximum velocity can only be inferred from the imprisoned air spaces already mentioned, and from the force with which a tree was driven through a shed, as shown by the dotted line in No. 4, Plate IX, where it penetrated the backing, the roof and the solid rock filled crib-work, knocking out a plumb post in its passage, and was sawn off 10 inches in diameter at the face of the crib. At some points the avalanches cross the valley and ascend the opposite slope to the extent of 200 or 300 feet. Sheds on a track located over 100 feet above the valley have been struck by avalanches from the opposite side which ascended the slope, passed over them, and climbed the mountain side, 150 feet above their roofs.

Remarkable effects are produced by the local cyclone or hurricane induced by the swift avalanches. This sometimes extends for 100 yards outside the course of the solid avalanche and is called the "flurry," because it is clouded with particles of fine snow. If the course of the avalanche is diverted by some natural obstacle, the flurry drives on in the line of original motion, snapping off huge trees several feet in diameter, at heights 50 feet or more above the ground, without uprooting them. Some in the vortex of the flurry are uprooted, but the majority are cut short off, as they would be by chain shot, and so far from the line of the avalanche that there is nothing to indicate the cause of their decapitation but the snow, impacted like moss against the windward side of their huge trunks. The flurry whirls upward to the height of 100 feet above the descending snow, and forward in advance of it when under full headway, presenting a magnificent spectacle to an observer at a safe distance. December last it picked up a man, and whirled and twisted him so rapidly and spirally, that when dropped he was a limp mass, without a bruise or break in skin or clothing, yet with all his bones broken or dislocated.

Bridges, which have been substituted for trestles carried away by slides, are anchored by guys to "dead men" in the ravine, and thus secured have successfully resisted the "flurry," which, although it caulked the chord spaces very tight with hard snow, did no damage to the structure.

With three-quarters of a mile addition to the snow sheds, Mr. R. Marpole, the experienced and capable Superintendent of the Pacific Division, is confident that interruption to traffic in the Selkirks will be limited to hours instead of days, as has been the case heretofore; and be chiefly the result of local damage to sheds from rocks or trees brought down by the avalanche.

The avalanche season—though a lively and brilliant one—is short in

comparison with the glacier one, and when the exposed points are all protected, interruptions due to the environment will have little appreciable effect upon the general traffic of the year. There will be an exceptional item for maintenance here in excess of any other division of the road, but I believe it will be fully met by the exceptional attractions of this glacier section. The avalanche may attract hundreds of bold admirers for two or three months of winter, but for the greater portion of the year the silent majestic glaziers, which may be approached without risk, will draw thousands of tourists into the Selkirk range, where there is no danger when there is no snow in motion.

Mud Slides.—A great deal of trouble has been experienced from mud slides and "gumbo" cuts, generally below the snow-shed level in the flanks of the Selkirks, and chiefly on their western or wettest slope. In forcing track laying many slopes were left too steep; but there are cases where the angle of repose, without any provocation, has proved to be a very obtuse and inconstant one, giving rise to acute expressions of disgust on the part of the roadmaster, and affording a signal illustration of total depravity in inanimate things. In comparison, the snow slide from above is clean and respectable, but the trouble in gumbo cuts is low in origin, of vicious proclivities and of the earth, earthy. No amount of cleaning is appreciated, and it requires to be sat upon. To effect this, the cut is deepened by steam shovel and derrick, and secured by a double row of piles on each side, 8 feet apart, the lower one (in the ditch) at 5-feet centers, the upper one (on the slope) of 3-feet centers. The inner row of piles is kept in position by a horizontal flatted sill across the track below subgrade, and the outer rows by similar but sloping braces between the tops of the inner and outer piles. A single log is run behind the ditch piles under the sloping brace, and a wall of about five logs high is carried up behind the outer row. The slopes are removed and coarse gravel is filled between and behind piling and in ditches, which permits all water to ooze through to the latter, where, owing to the grade, it readily gets away.

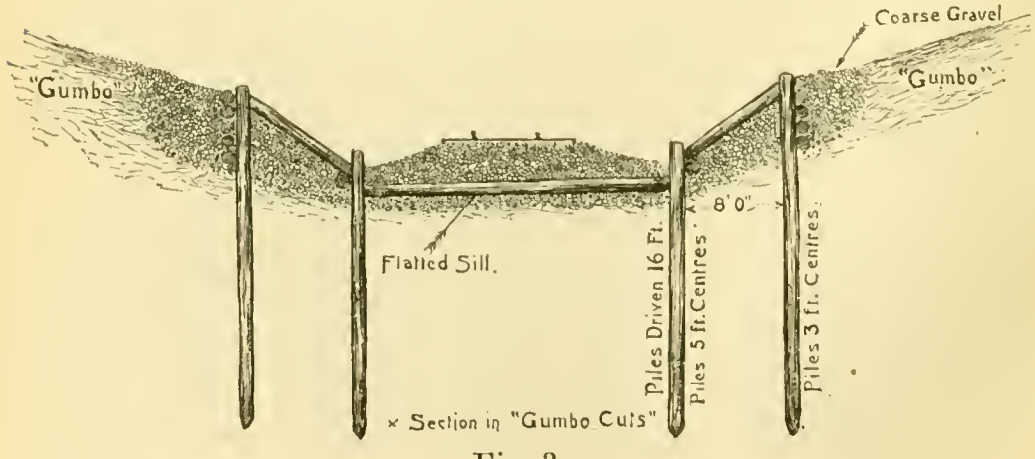


Fig. 3.

The "gumbo" of the Selkirk slopes is not the material called by that name in California, where it is found upon the surface of level plains, and bakes and cracks with the sun, but becomes a sticky mud with every rain. The Selkirk material is a sandy loam quick sand, and would be steady enough if it had not imbibed so much—water.

The sketch on opposite page explains the manner in which it has been successfully dealt with in the cuttings.

Bridging.—All the bridging in the mountain section was at first necessarily of wood, which is abundant and of excellent quality there. Some of these have already been replaced with steel. The section built by the Government at the Pacific end of the road—with the exception of the cantilever across the Fraser—has wooden bridges of the Howe truss pattern. In the Selkirks there are three high bridges—154, 175 and 294 feet in height respectively. The last, the Stony Creek Bridge, is 490 feet in length, the greatest span being 172 feet, resting on wooden towers 200 feet high, standing on a concrete foundation. It is probably the highest wooden bridge in America. It is soon to be replaced with a steel arch springing from the rocky sides of the V-shaped ravine, about half way of its depth; and the other two high ones, with iron trestles. The metal bridges erected by the Company, east of the Rocky Mountains, are of heavy pattern, designed by the late C. Shaler Smith, M. Am. Soc. C. E. Where through trusses are unavoidable, these have a width of 20 feet between centers.

Besides much that has been filled in, there yet remains a large amount of trestle work east of Lake Superior and upon the Government section at Pacific Coast, which has been substantially built and floored, as it will under any circumstances require years to substitute permanent work for them. The Company has bound itself to expend the whole of the $15 000 000—interest on which is guaranteed by the Government for fifty years, in consideration of the abandonment of their monopoly—upon the main line between Quebec and Vancouver. Of this amount five and a quarter millions is apportioned in the agreement to rolling stock, five and a half millions on capital account to "buildings, snow-sheds, sidings, permanent bridges, filling in trestles, reducing grades and curves, and other improvements." The remaining four and a quarter millions is apportioned to "elevators, bridges, locomotive shops, filling trestles, sidings, docks, and lake and coast steamers."

By crossing the Selkirks instead of going round them in the Columbia River Valley, the road is shortened about eighty miles. The fall in the Columbia River between the first and second crossings (going westward) is 1 100 feet, an average of about 7 feet per mile. The river has its cañons, and in places washes the base of the mountains, so that heavy work and possibly some tunneling would have been encountered on the longer route.

On leaving the Columbia at the second crossing, and where it soon ceases to be a Canadian river, the line crosses the Gold Range through the Eagle Pass, a remarkably favorable one, the summit being only 1 800 feet above tide, although in a range with many snow-capped mountains. There are nine snow-sheds, with a total length of 1 360 feet, all on the western slope of the Eagle Pass. From the western side of the Gold Range, the line follows the shores of lakes and rivers which discharge into the Pacific Ocean upon Canadian soil. In crossing the Dry Zone or bunch-grass grazing plateau of British Columbia, there is heavy work and tunneling along the rock-bound shores of the lakes; but it is when the line descends the Thompson and Fraser Rivers, where these cut through the Coast Range, that the heaviest consecutive hundred miles on the whole route is encountered. This section, built by the Government, cost about $10 000 000, or $80 000 per mile, without rolling stock or stations. There are numerous tunnels and rock cuts, as well as heavy earth cuts, and a fine cantilever of 300 feet span across the Fraser River, which was the second erection of the kind in America, and was designed by Mr. C. C. Schneider, M. Am. Soc. C. E.

THE EQUIPMENT OF THE LINE.

The Canadian Pacific Railway is a modern road, having had the utmost freedom of location in unoccupied territory, for stations, yards and shops, and has not therefore been handicapped by costly accumulations of antiquated rolling stock, or hampered by limited yard accommodation on any portion of the contract route—conditions which have proved so onerous to some older roads.

The divisional points are placed as nearly as possible at intervals of 125 miles, any variation from this being due to the questions of suitable station ground, or water supply. At these points, the tracks are arranged as shown on the standard plan, the object being to provide a yard that may be readily extended—one in which the main track is broken as little as possible by switches, and so arranged that any car in the yard may be reached by one shunt.

At alternate divisional points, shops are established of sufficient capacity for repairs of rolling stock on two working sections, and at the divisional points between these there are smaller shops with the few necessary tools for ordinary breakages. Engines run from the larger shops to the smaller ones, so that ordinarily they return to the principal shop points every other day.

At all divisional points, the water tanks are erected 40 feet high, to give a sufficient pressure for washing out engines.

At the alternate points, wrecking cars, pile drivers, tool cars, bridge and track material, are provided for any emergency on the sections either way from them, and a smaller supply of emergency material is kept at the intermediate points.

In the newer country, stations are arranged at intervals of about 16 miles, governed by ground and water supply, with accommodation for two section gangs of eight men each, a combined freight and passenger station, a 50 000-gallon tank, and a telegraph office, ensuring collection of section gangs for any emergency in the shortest possible time. These regular stations have side tracks according to trains handled on the division, and, where local traffic exists, a business track as well as a passing track. Passing tracks are laid about half way between these stations, making the crossing interval generally eight miles; but this is reduced where there is considerable traffic.

At Montreal, the principal eastern terminus; at Vancouver, the Pacific terminus; and at Winnipeg, which is midway between them and has 24 miles of sidings, large shops exist for heavy repairs of cars and locomotives. As these three points are 1 500 miles apart, large intermediate shops will be required as traffic increases.

Fuel Supply.—The fuel supply is: Nova Scotia coal for the eastern system, which is carried a short distance west of Ottawa; Pennsylvania coal from this point to Brandon, on the prairies, the first divisional station west of Winnipeg. This coal is brought by rail across the St. Lawrence and Niagara Rivers, and by water to Lake Superior. West of Brandon, Canadian tertiary coal from the Bow River deposit is used, until it is met in the mountains by the Pacific Coast coal from Vancouver Island. The Bow River coal is estimated to be within fifteen per cent. of the value of Pittsburgh coal. Anthracite is being worked alongside the main line in the Rocky Mountains and is used for passenger cars and domestic purposes as far east as Winnipeg, but export is as yet chiefly to San Francisco. When more extensively mined and fire boxes are altered to burn it, it may displace other coal in the mountain section.

Windmills have proved successful for pumping on the prairies. The water is prevented from freezing by a heating pipe passing up through the center of tank.

Locomotives.—The consolidation engines working the Selkirk Division were built at the Company's shops at Montreal. They are distinguished by their short stroke, 22 inches, high boiler pressure, 160 pounds, and large grate surface to maintain this pressure. Their weight, 94½ tons, is sufficient to prevent slipping in good weather, when hauling full train of seven coaches, without the use of sand, but this is provided both front and back for bad weather. The tractive force is 155.7 pounds per pound pressure on pistons, the wheel base short in proportion to diameter of drivers, and being carefully counterbalanced, they run with speed, ease and steadiness around sharp curves. Their brake power is the Westinghouse on two forward pairs of drivers, and the American

Steam Brake on the two hind pairs. The Water Brake is also applied to all engines running in the mountains. The automatic brake is used in ascending, and straight air in descending, with hand brakes manned. The Block system, with telephone addition, is extensively used in the mountains.

The principal dimensions are:

Diameter of cylinder and length of stroke, 19 x 22 inches.
Distance apart of centers, 6 feet 11 inches.
Length of connecting rod, 9 feet 2 inches.
Driving wheels, diameter, 4 feet 3 inches.
Driving wheel tires, width and thickness:
First and fourth, 5½ x 3 inches, flanged.
Second and third, 6 x 3 inches, blind.
Fixed wheel base, 14 feet 3 inches.
Total wheel base of engine, 21 feet 3 inches.
Center of cylinder to center of driving axle, 13 feet.

Weight on track in working order.....	13 100	pounds.
Weight on drivers in working order.....	90 900	"
Total weight of engine.....	104 000	"
Weight of tender, empty.....	35 000	"
Capacity of " coal.....	20 000	"
" " water.....	30 000	"
Total engine and tender, in working order,	189 000	"

Snow Ploughs.—In winter these consolidation engines are furnished with a large heavy pilot plough, shown in drawing, Plate IX. This plough has rendered excellent service and has repeatedly opened the way through packed and saturated snow, where the large wing plough had failed, enabling the latter to follow with wings wide open, nose down and flanger working, securing a good rail. These ploughs are of $\frac{5}{16}$ iron, double plated at nose, steel angles, and 6-inch by 1-inch iron strap stays. The height of nose is 5 feet and of wings at ends 7½ feet, clearing a width of 9 feet at bottom and 10 feet at top.

The regular snow train has a strongly built plough, wings 16 feet across, and nose 11 feet above rail, the lower or horizontal portion of which is raised or lowered from inside, and when pressed down by weight of snow, is carried by rollers running on top of rails. The flanger is adjusted to turn over on meeting any obstruction harder than ice or packed snow. For this train Y's are put in, through which the whole train can be turned and see-saw back and forth, giving no rest to the wicked drifts or slides. "*Principiis obsta!*" is, during snow storms, the motto on the crest of the Selkirks.

For the efficient working of the snow plough train, it has been found necessary in many places, and where possible, to remove the line out from the hillside, to leave room for the accumulation of snow on the slopes, and a chance for the inside wing of the plough. This consideration is apt to be overlooked during a summer location in a mountainous, snow affected region, especially when working against time, or upon too economical lines. The teachings of experience in the Selkirks have been many and valuable, and none more so than this, the question of sea room for the plough, and of store room for the snow.

The freight engines are heavier and more powerful than the passenger ones; cylinders 20 x 26; 4 pairs of 48-inch drivers; wheel base, 21 feet 11 inches; driving wheel base 14 feet; weight, 116 000 pounds; weight on drivers, 102 000 pounds. These engines haul 12 loaded cars up the Selkirk slope, which has grades of 116 feet per mile. In descending long heavy grades with these trains, frequent stoppages are made to cool off, and prevent breakages in the cast-iron plate wheels. This precaution is not necessary with passenger trains, in which no cast-iron wheels are used.

On the Selkirk division steel rails of 72 pounds weight per yard are used, with 3 500 ties per mile.

Provision Magazines.—The Company have omitted no precautions to secure the safety and comfort of passengers. For hundreds of miles no supplies can be procured except by train, and in view of detentions, each through train from Montreal, in addition to the dining-car supplies, carries, in the baggage car, an emergency box of provisions, to be used exclusively for passengers, and only in case of necessity. Besides this, at nine points on the Selkirks and Eagle Pass, where detention by snow slides is possible, provision magazines are established in safe positions, at intervals of about ten or twelve miles; so that no train may be caught more than six miles from food. These provisions are emptied in the spring, and replenished with fresh supplies in the autumn. Coal and oil supplies for the passenger cars are also similarly "cached," and emergency fuel for the locomotives, bridge and track material are held loaded on cars, to shorten detention of trains.

Extremes meet — the voyageurs of the Hudson Bay Company, Arctic explorers, and the hunters and trappers in the mountains, cached their surplus stores against the ravages of fire, of the loup cervier, the wolverine or the polar bear; and now the most recent specimen of the highest type of transportation confirms, by its emergency magazines, the wisdom of the pioneers in the old time before the railway era.

PROSPECTIVE TRAFFIC.

The Canadian Pacific Railway has been opened for traffic through 2 500 miles of territory almost uninhabited, and so rapidly that settlement could not keep pace with it. The 1 900 miles and over of main line constructed by the Company has been built in half the time allowed by the contract, and within these five years a subsidiary system, about 2 300 miles in length, has been built or acquired, by which the main transcontinental line has already been made more than self-sustaining. The capital account is not yet closed; another five years will be required to convert temporary into permanent work, and new demands will arise from extension of traffic, both on the prairies and in the mountains. Under these circumstances, no adequate conception of its importance can be formed without some consideration of the character of the country it traverses, and upon which its future depends.

The Eastern Section.—The starting point of the national road, as a Government work, was a point near Lake Nipissing, called Callander, about equally distant from Ottawa and Toronto, and about two hundred miles due north from the latter. This point had no connections, and no special merit but that of being equally inconvenient to the rival Provinces of Ontario and Quebec, both of which were placed on equal terms in reaching it with their provincial lines. One hundred miles west of Callander is Sudbury, the junction of the important line from St. Paul and Minneapolis, *via* Sault Ste. Marie, by which those cities find their shortest all-rail route to Atlantic tide-water. Sault Ste. Marie is rather nearer to Montreal than Detroit is, with the advantage that, like the Niagara and St. Lawrence rivers, its broken navigation makes it a bridge route.

It is not necessary to refer to the country east of Sudbury, which is a lumbering, agricultural and mining region, quite capable of sustaining a railway, even without the traffic of the Sault route. From Sudbury westward, the line cuts through continuous forest for 360 miles, until it strikes the shores of Lake Superior, which it skirts for 200 miles, and then leaves in a very direct line through a forest and lake region for the outlet of the Lake of the Woods, nearly three hundred miles farther west. The 850 miles from Sudbury to Lake of the Woods is through a country of similar character—a mountain and lake region—with very limited arable areas, but very promising mineral ones, and with an immense supply of timber invaluable to the railway, but much of which is, at present, commercially beyond the reach of market. Innumerable lakes, some more than 20 miles in length, are tapped by the railway, which, with their thousands of miles of coast line, will yield valuable supplies of timber, as soon as the nearer ones are exhausted, or the price makes exportation profitable. The 200 miles shore line on Lake Superior,

with excellent harbors at the extreme points, afford landing places for water-borne coal from Ohio and Pennsylvania, for carriage east and west, as well as reach the valuable fisheries of the coast.

Near Sudbury and Port Arthur, gold, silver, copper and iron have been discovered, and, with the exception of the iron, are being worked. Upon the extension of these discoveries, in what is all known to be a mineral region, between Sudbury and the Lake of the Woods, as well as upon the lumber trade, the building up of a local traffic will chiefly depend.

The Lake of the Woods has an area of 700 square miles at an elevation of 1 062 feet above tide water. Its drainage area is about twenty-five thousand square miles—7 000 miles of which are in Northern Minnesota—which its water-shed penetrates to the head waters of the Mississippi at Lake Winnipegoshish. In Canada its water-shed begins within 30 miles of Lake Superior. It discharges into Winnipeg River with a fall of 21¼ feet, at the foot of which the river turns abruptly westward and runs for three miles parallel with the lake shore, and separated from it by a narrow natural dam of rock, through which, at half a dozen points, the lake waters can be conducted by a flume of 100 yards in length. The estimated water power at this dam is 65 000 H. P., and between it and Lake Winnipeg the river has a fall of about three hundred feet. The railway line follows this dam, and saw-mills with cutting capacity of 60 000 000 feet B. M. per annum, working twelve hours daily, are in operation, for the supply of the prairie region as far west as Regina, nearly five hundred miles, where it meets the timber and lumber from British Columbia. It is estimated that the timber supply from this point is good for thirty years, at double the present rate of consumption.

A flour mill, of 1 200 barrels daily capacity, has recently been erected at Keewatin. There was a surplus wheat crop in Manitoba last year, exceeding ten millions of bushels, grown within an average haul of 250 miles of Keewatin. Thus there is already a possible wheat growth sufficient for half a dozen such mills. There are already forty-four elevators at way stations, with capacity of over two million bushels, and a still larger storage capacity for these on Lake Superior. The wheat elevators already extend more than 300 miles west of Winnipeg.

The Plain and Prairie Section.—The Rocky Mountains, which, from Santa Fé, in New Mexico, to Cheyenne, in Wyoming, run due north through Colorado on their most eastern projection, turn at Cheyenne (longitude 105 degrees west from Greenwich, or 28 degrees west from Washington), running northwest to the International Boundary, and at Calgary (the Canadian Denver) strike the 115th meridian (38 degrees west from Washington), carrying the plain and prairie regions 10 degrees farther west than they are in Colorado. The width

of the Canadian fertile belt west of the Red River, is about the same as that of the prairie regions between Indiana and Colorado.

The prairie section, according to the Canadian Geological Survey reports, may be said to extend from the Red River on the 97th meridian west from Greenwich, to Calgary near the Rocky Mountains on the 114th meridian, a distance of 800 miles, and from the 49th to the 54th degrees of north latitude. There are three distinct plateaux or "steppes," sloping from the Rocky Mountains northeasterly towards Lake Winnipeg and the Red River, having well-defined escarpments running northwesterly parallel with the range. The general slope from the foot hills of the Rockies averages about five feet per mile. The lowest of these plateaux averages about eight hundred feet above the sea, and embraces an extensive lake system nearly fourteen thousand miles in extent, the largest (Lake Winnipeg) covering 8 500 square miles. The total area, including the lakes, is 55 000 square miles. This interior basin, the lowest of the continent, generally known as the Red River Valley, has the finest wheat land perhaps in the world. It is only 52 miles wide at the International Boundary, and rises thence southward for about two hundred miles, attaining an elevation nearly one thousand feet above sea level.

The second steppe is about two hundred and fifty miles wide at the 49th parallel, and 200 miles at the 54th, having an area of over 100 000 square miles, 71 000 square miles of which form the eastern portion of the Great Plains. Its average elevation is 1 600 feet above sea level.

The third steppe has an average elevation of 3 000 feet, being 4 000 feet at the foot hills and 2 000 feet at its eastern edge. Its area is 134 000 square miles, of which 115 000 are almost entirely devoid of forest. Its breadth on the 49th parallel is 465 miles.

The total area south of the 54th parallel is 280 000 square miles—about one hundred and eighty millions of acres—of which, after allowing for swamps and lakes, mountains and barrens, by far the greater portion is arable.

The agricultural capabilities of the Canadian northwest are not, however, limited by the 54th parallel. That latitude is the northern boundary of the great plain and prairie region, which extends from Mexico through the United States to the Great Saskatchewan. Narrowing northward of the Winnipeg Lake basin, by the encroachment of the Laurentian formation on its eastern border, it extends as broken prairie and wood land to the shores of the Arctic Ocean, where its breadth is reduced to between three hundred and four hundred miles. Beyond the North Saskatchewan River, it loses its essentially prairie character, and from increasing moisture of climate becomes generally thickly covered with coniferous forest. From the best estimates which can be made in this imperfectly explored country, it is believed that it contains at least 120 000 000 of acres of arable and pasture land north of the 54th parallel.

Thus there is in the Canadian northwest about three hundred millions of acres of arable and pasture land, of which one-third or more may be capable of producing wheat of the finest quality known.

In a recent report of the Senate of Canada, it is stated that this northern forest-covered region embraces also the greatest fur producing country in the world, supplying three-fourths of all the valuable furs sold in Leipsic and London, to the annual value of millions of dollars.

The climate of the eastern slope of the Rockies, for a belt of over one hundred and fifty miles in width is, as compared with the plains on the same latitude eastward, exceptionally mild in winter. A southwest wind called the "Chinook," blowing at right angles to and over the Rockies, brings a thaw, removing snow and enabling cattle to feed out all the year round. At Canmore, in the Rockies, 4 200 feet above tide, cattle range out all winter. The remarkable warmth of a wind flowing for hundreds of miles over snow-covered mountains, could not be accounted for by the proximity of the warm waters of the Pacific, and is explained by the alternate expansion and condensation of air flowing from the ocean level over the mountains, and descending thence to the plains below. As the moisture is evaporated, or the air expanded, in rising over the mountains, latent heat is absorbed which is given out again by the condensation of the moisture or the compression of the air in descending to the plains below.

Another climatic feature peculiar to all high latitudes, which, according to Dr. Dawson, of the Canadian Geological Survey, is believed to account for the ripening of grain and vegetables in the Peace River region and north of the 60th parallel, is the greater length of the day and the greater amount of sunshine, the sun rising on June 21st at three hours and twelve minutes and setting at eight hours and fifty minutes.

The Bow River coal area is estimated to contain 330 000 000 of tons, and will be the chief source of supply for the prairie region and for many hundred miles of the railway, and an increasing source of traffic for the latter.

Natural gas has been discovered in boring for water near the foot hills, and is used for pumping at two of the Company's stations.

The Mountain Region.—The Rockies, where crossed by the Canadian Pacific Railway, are separated from the Selkirks (one of the Gold ranges), by the Columbia River flowing north; and the Selkirks from the Gold Mountains, by the same river flowing south. Between the gold and coast ranges lies an undulating, bunch grass region known as "the dry zone," one of the finest grazing districts in Canada, but where crops require irrigation. This interior plateau has an average width of 100 miles, and an average elevation of 3 500 feet. There is an excessive rain fall on the coast, averaging about six feet per annum, falling chiefly be-

tween October and April, and a very great precipitation, particularly of snow, upon the Selkirks. The rain clouds from the Pacific being elevated by the coast range (with its higher peaks of 6 000 to 7 000 feet average, and some exceeding 9 000 feet), pass over the interior plateau, and precipitate their stores of rain or snow upon the Gold Mountains, and chiefly upon the highest of these, the Selkirks. The Rockies, therefore, although the loftiest of all the ranges, are the driest, and no snow-sheds are required in them.

The agricultural resources of British Columbia are limited chiefly to this interior plateau and to the delta of the Fraser River. Lumber, fish and minerals, in each of which her resources are unsurpassed, are the great features of the Pacific Province, and these industries will furnish a local market for her agricultural products, her exports of which will, until irrigation is extended, be confined to horses and cattle. Through the Rockies, Selkirks and Gold Range, the railway has penetrated a hitherto inaccessible region and opened up a virgin forest, in which Douglas fir (Oregon pine) and cedar abound, with spruce and various pines. The first two are timbers of such value that they will bear rail transport to the northern Atlantic coast, where we have nothing to compare with them. Lumber is now exported from British Columbia to Japan, China, Australia and South America. Since the commencement of the railway, saw-mills have been established at eight different inland points in the mountains.

Coal and iron abound at tide water, as well as in the Rockies—the former the best in quality yet found on the Pacific coast, half a million of tons of which are now exported annually. Fifty millions of dollars in gold have been washed out in the Province in the last thirty years, and quartz mining is now becoming a result of the railway. Silver is mined of sufficient richness to bear transport to a smelter at Omaha, a carriage of 1 750 miles. Smelting works and sampling mills are now being erected with the assistance of the Provincial Government.

Over 3 600 tons of canned salmon are exported, nearly all to Great Britain. The railway has opened an eastern market for this fish in its fresh state.

The railway has created a terminal city, which will soon surpass the older ones of Victoria and New Westminster, and has given an impetus to the coasting trade which reacts on itself; so that, with the wonderful natural resources of this Province, the commercial success of the road in its freight traffic is already assured in the mountain region, where so little was expected at the first, that security was required by the Government for ten years' continuous operation of the road.

The Asiatic commerce is yet in its infancy, under temporary arrangements with chartered steamers. When the subsidized steamers now under construction for this trade are put on the route, both freight and passenger traffic may be expected to assume important proportions.

Lumber and flour are principal articles of export; teas, silks and curios, of import. Both New England and Canadian cotton manufactures have been exported to China by this route.

For local passenger traffic, which from the sparseness of population has, like the freight business to be created, there is chiefly that of prospectors for minerals and timber, ranchmen, miners and lumbermen, and settlers in the new towns, which can be regarded as tributary to the road. Through traffic with all the Pacific coast is competed for, and tourist travel is specially cultivated. For this, the route through the mountain region offers exceptional attractions, and no expense has been spared to make the most of this class of traffic. The hotels at the National Park in the Rockies, and at the terminus, Vancouver, are, like all the Company's equipment, modern and complete. The scenery is Alpine, the route the only glacier one in America, and comfortable hostelries have been established in the mountains for stop-over tourists or sportsmen wishing to hunt the Rocky Mountain goat, now about limited to these latitudes, the big horn, the grizzly and the mountain lion ; or, farther north, the cariboo ; and in the foot hills, deer, elk and antelope ; or to cast a fly in the trout streams and lakes of the mountain regions.

CANADIAN PACIFIC RAILWAY.

Condensed Balance Sheet, December 31st, 1887.

Cost of Road.				**Capital Stock**		$65 000 000 00
Main Line (including dividends paid, less net earnings to date)	$116 618 039 28			**Mortgage Bonds.**		
Acquired Lines	12 665 205 53			First Mortgage 5 per cent., £7 191 500	$34 998 633 33	
Branch Lines	2 850 253 59			Canada Central First Mortgage, amount of issue £500 000 $2 433 333		
Algoma Branch	3 556 131 11			Sinking Fund deposited with Dominion Government 1 500 000		
		$135 689 629 51		Amount necessary to redeem balance at maturity	850 000 00	
Note.—Lines built by Government not included. Estimated cost, $35 000 000.				Canada Central Second Mortgage Bonds, 6 per cent	973 333 33	
Equipment.				Due Province of Quebec on Q., M. O. and O. Ry., 5 per cent	3 500 000 00	
Rolling stock	$9 107 067 95			Due Province of Quebec on North Shore Ry., 5 per cent	3 500 000 00	
Lake steamers	478 952 99			North Shore Railway First Mortgage Bonds	436 053 00	
Shops and machinery (Montreal, Hochelaga, Perth and Carleton Place)	984 912 28					44 258 019 66
		10 570 933 22		**Land Grant Bonds, First Mortgage.**		
Construction plant, tools and outfit		124 702 63		Amount of issue $25 000 000		
Real estate (at and near Montreal)		283 785 80		Less amount redeemed and cancelled 16 529 000		
Advances and Expenditures on Leased Lines.					$8 471 000 00	
Southeastern Railway	$1 604 780 53			Amount held in trust by Dominion Government and not bearing interest $1 000 000		
St. Lawrence and Ottawa Railway	296 414 34			Amount held by Company 4 000 000		
Ontario and Quebec Railway	1 679 363 40				5 000 000 00	
	$3 580 558 27					3 471 000 00
At credit of Atlantic and N. W. Ry. (amount in hand for completion of line)	1 780 271 10			**Current Liabilities.**		
		1 800 287 17		Including vouchers and pay rolls and amount advanced in anticipation of sale of Bonds on Algoma Branch	$7 036 820 69	7 036 820 69
Station balances, accounts receivable, and miscellaneous securities outside of capital account		2 615 323 29				
Material and supplies on hand		1 885 535 12		Note.—Algoma Branch First Mortgage Bonds, £750 000 in Treasury, sold early in 1888, realizing $3 467 500.		
Dominion Government Guarantee Fund.						
Amount on deposit to guarantee 3 per cent. per annum on capital stock until August 17, 1893		10 310 954 75				

BALANCE DUE ON LAND SOLD.			INTEREST ON FUNDED DEBT AND RENTAL OF LEASED LINES.		
Deferred payments		1 252 857 10	Coupons not presented, including amounts due January 1, 1887	$1 015 361 24	
Cash in treasury		1 014 802 11	Accrued to date, not due	234 827 82	1 250 189 06
NOTE.—14 934 237 acres of land unsold, valued at $2 per acre, $29 868 474.			CASH SUBSIDY PAID BY DOMINION GOVERNMENT		25 000 000 00
			LAND GRANT.		
			3 272 749 acres sold, amounting to	$10 236 541 45	
			Less expenses, rebates and 10 per cent. premium on L. G. Bonds taken in payment	1 811 368 81	
				$8 425 172 64	
			6 793 014 acres taken by Government, to $1.50 per acre	10 189 521 00	18 614 693 64
			TOWN SITES.		
			Amounts received from the sale of Town Sites not covered by Land Grant Mortgage	$915 342 95	
			Less expenditure grading streets, buildings, etc., at Vancouver	304 855 30	610 487 56
			Bonuses received from Municipalities		307 660 00
		$165 548 810 70			$165 548 810 70

(Signed) I. G. OGDEN,
Comptroller.

The illustrations with this paper are:

A general map of the line of the Canadian Pacific Railway. Plate I.

A map of that part of the line between Beaver Station and Craigellachie, showing location of snow sheds. Plate V.

A topographical map of the line in the vicinity of the summit of the Selkirk range. Plate VI.

A profile of that part of the main line between Calgary and Sicamous, showing all heavy grades between the Atlantic and Pacific, and also the location of the snow sheds. Plate VII.

Plan of a standard divisional yard. Plate VIII.

Plate showing the various types of snow sheds, glance fence and split fence—also consolidation engine, tender and snow plough. Plate IX.

Three photographs of snow sheds, showing also overflow of an avalanche at shed portal, and a deep snow cut at summit of Selkirks. Plates II, III, IV.

Cut showing method of blasting snow slide.

Cut showing excavation of snow slide filled with trees.

Cut showing method of treating "Gumbo" Cuts.

CRAIGELLACHIE
1130
1135
EAGLE
RIVER
Crazy Cr.
1125
1120
GRIFFIN LAKE
GRIFFIN LAKE
1115
VALLEY LAKE
CANWILLIAM
Summit Lake
SUMMIT OF GOLD RANGE
1110
COLUMBIA R.
REVELSTOKE
1105
1100
1095
Length of Snow Sheds = 1361 ft.
SCALE
1 2 3 4 5 Miles

LAN PACIFIC RAILWAY

Plan

PART OF MAIN LINE FROM

STATION to CRAIGELLACHIE

SHEWING LOCATION OF

SNOW SHEDS

PLATE V
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NOTE - Snow Sheds are in Red

Nº 1 = 1850 ft long	Nº 19 = 2017 ft long	Nº 37 = 300 ft long
2 = 860 "	20 = 1688 "	38 = 815 "
3 = 795 "	21 = 195 "	39 = 141 "
4 = 650 "	22 = 610 "	40 = 541 "
5 = 205 "	23 = 200 "	41 = 832 "
6 = 432 "	24 = 1030 "	42 = 159 "
7 = 2850 "	25 = 110 "	43 = 152 "
8 = 1185 "	26 = 80 "	44 = 160 "
9 = 136 "	27 = 80 "	45 = 97 "
10 = 1784 "	28 = 500 "	46 = 97 "
11 = 296 "	29 = 328 "	47 = 81 "
12 = 774 "	30 = 104 "	48 = 477 "
13 = 1003 "	31 = 120 "	49 = 49 "
14 = 1016 "	32 = 500 "	50 = 297 "
15 = 424 "	33 = 460 "	51 = 105 "
16 = 352 "	34 = 402 "	52 = 177 "
17 = 3098 "	34 = 120 "	53 = 121 "
18 = 266 "	36 = 355 "	

Total length of Snow Sheds = 31764 feet
or 6 miles fully

Length of Snow Sheds = 30403 feet

PLATE VI
TRANS. AM. SOC. CIV. ENG'NS.
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PACIFIC RAILWAY.

MAP

SHEWING THE

CANADIAN PACIFIC RY

IN THE VICINITY OF SUMMIT OF

SELKIRK RANGE.

SCALE 1 1/2 IN = 1 MILE

ENGRAVED BY OLIVER DONALD
J.E. GRIFFITH

CANADIAN PACIFIC RAILWAY.

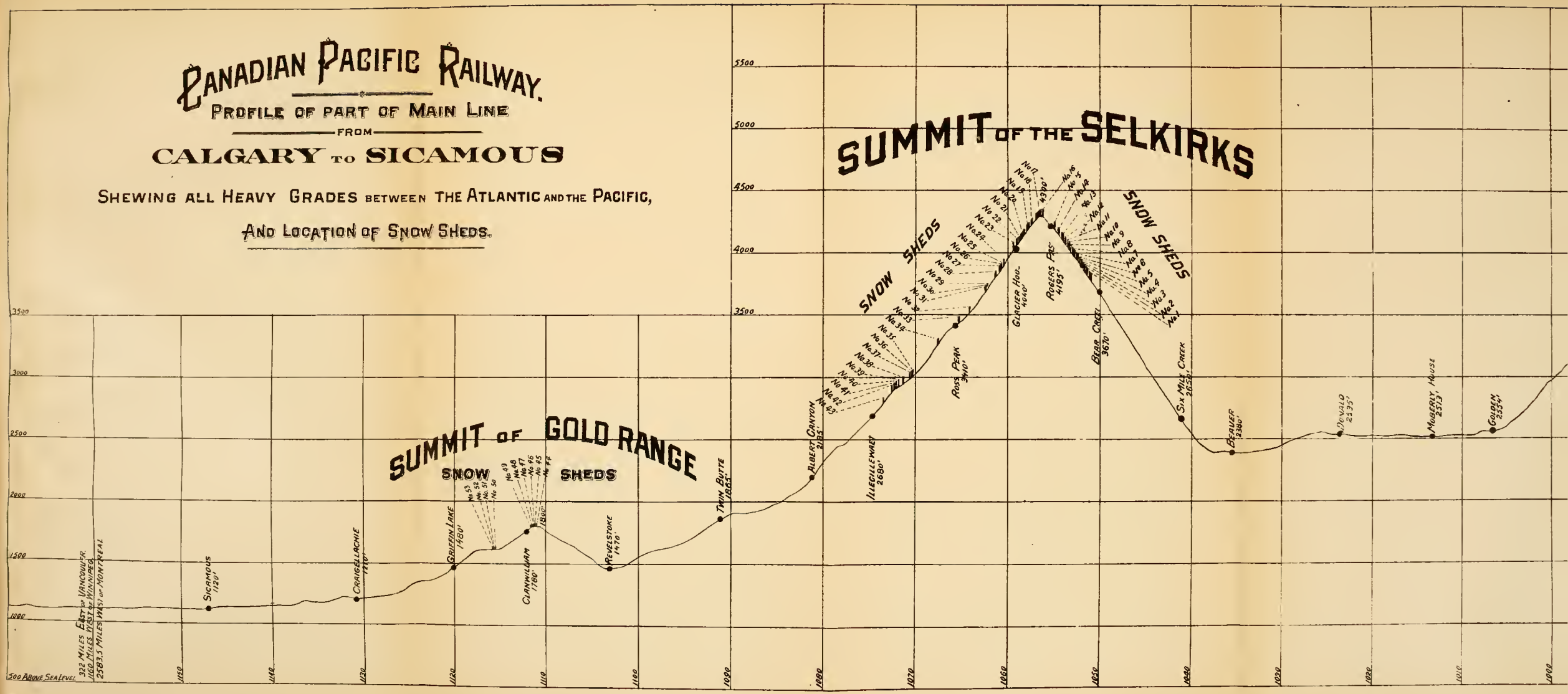
PROFILE OF PART OF MAIN LINE

FROM

CALGARY TO SICAMOUS

SHEWING ALL HEAVY GRADES BETWEEN THE ATLANTIC AND THE PACIFIC,

AND LOCATION OF SNOW SHEDS.



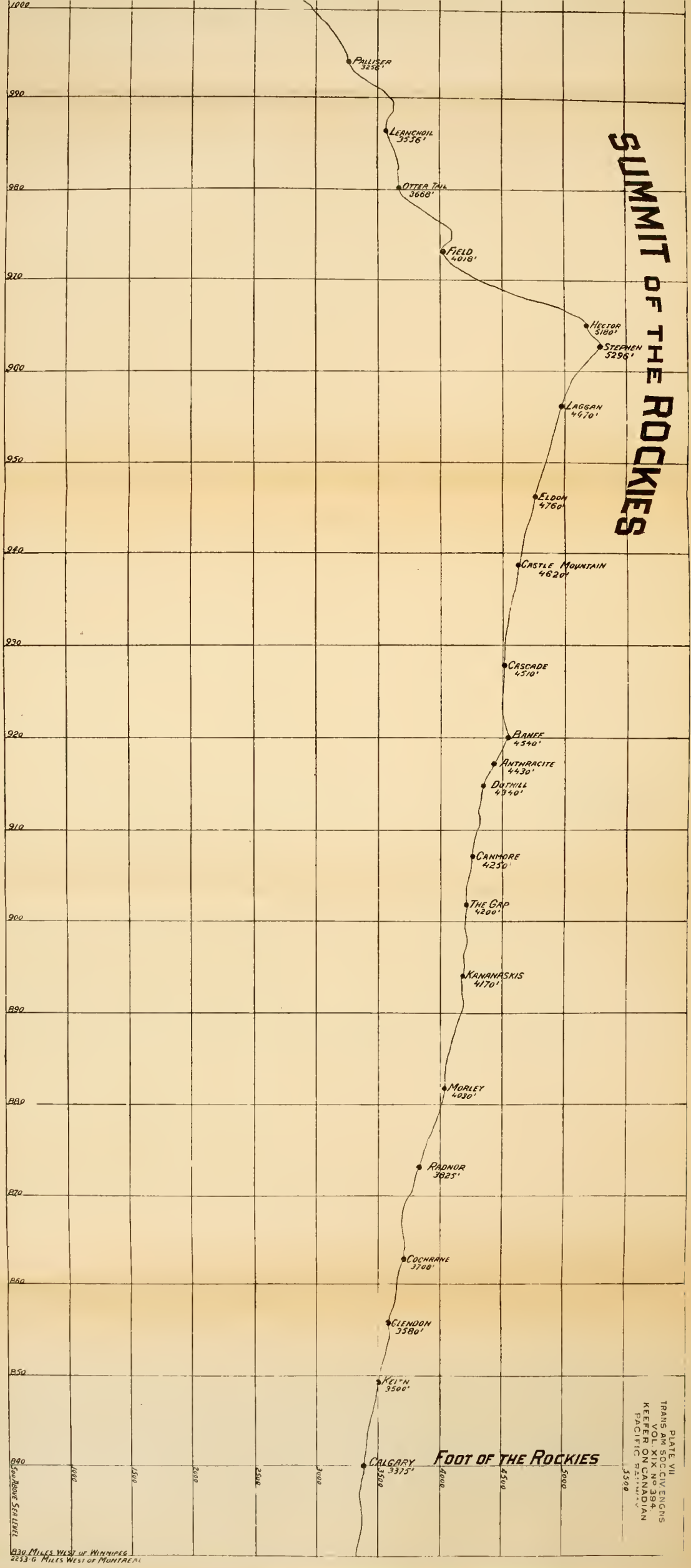

SUMMIT OF THE ROCKIES
PALLISER 3258'
LEANCHOIL 3536'
OTTER TAIL 3668'
FIELD 4018'
HECTOR 5180'
STEPHEN 5296'
LAGGAN 4670'
ELDON 4760'
CASTLE MOUNTAIN 4620'
CASCADE 4510'
BANFF 4540'
ANTHRACITE 4430'
DATHILL 4340'
CANMORE 4250'
THE GAP 4200'
KANANASKIS 4170'
MORLEY 4030'
RADNOR 3825'
COCHRANE 3708'
GLENDON 3580'
KEITH 3500'
CALGARY 3375'
FOOT OF THE ROCKIES
1000
990
980
970
960
950
940
930
920
910
900
890
880
870
860
850
840
Feet Above Sea Level
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
830 Miles West of Winnipeg
2253·6 Miles West of Montreal
PLATE VII
TRANS AM SOC CIV ENGNS
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PLATE VIII
TRANS AM SOC CIV ENGNS
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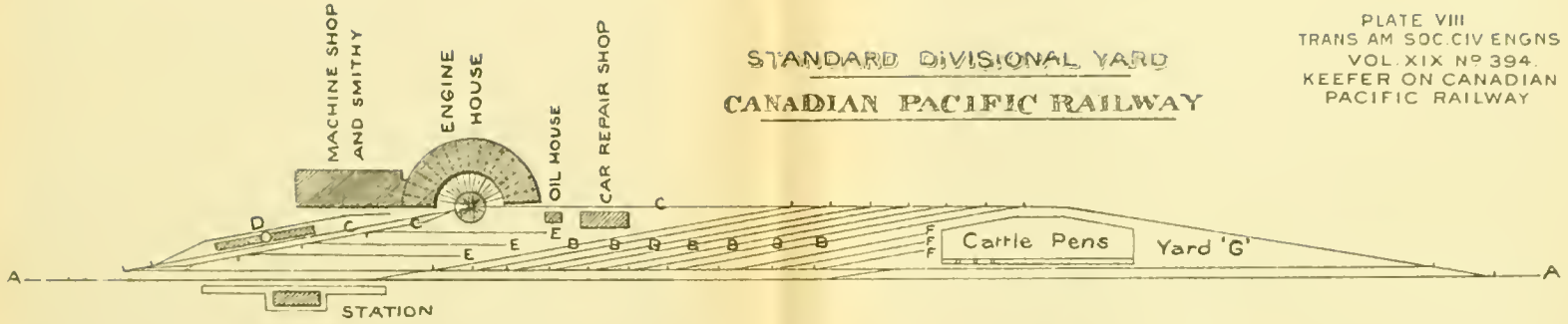
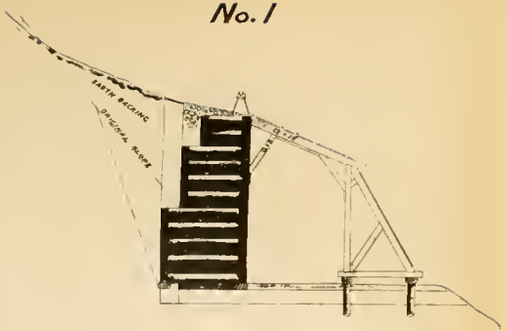
KEEFER ON CANADIAN
PACIFIC RAILWAY

STANDARD DIVISIONAL YARD

CANADIAN PACIFIC RAILWAY

—— REFERENCES. ——

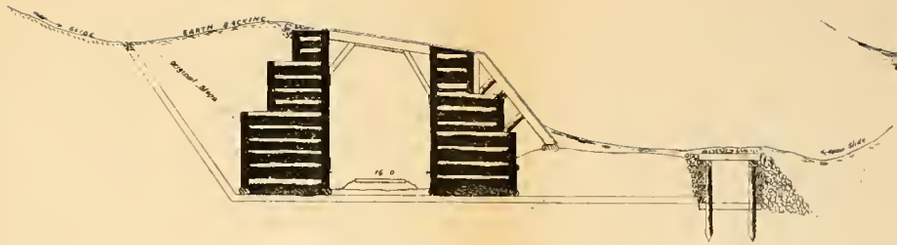
A. A. *Main Track.* — B. B. B. *Train Tracks.* — C. C. C. *Engine House Tracks.* — D. *Coal Track.* — E. E. E. *Car Repair Tracks.* — F. F. F. *Tracks for Pile-driving Cars, Derrick Cars, Snow Plows and other Emergency Equipment.* — G. *Yard for Piles, Bridge Timber and Emergency Materials.* ——

No. 1



TYPICAL SHED.

No. 2

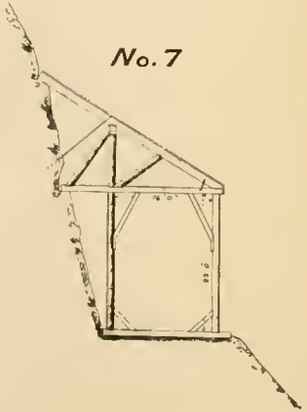


VALLEY SHED.

No. 3

TYPICAL SHED.

No. 7



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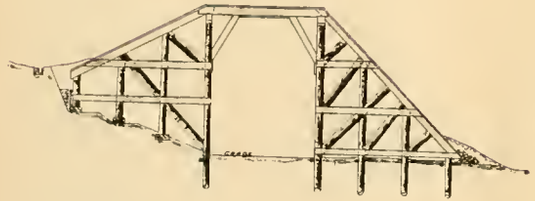
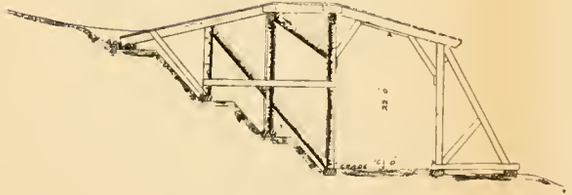


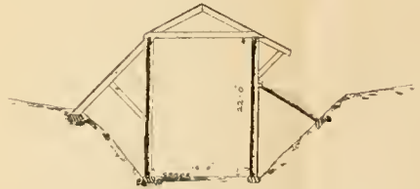
GALLERY

No. 9



SHEDS

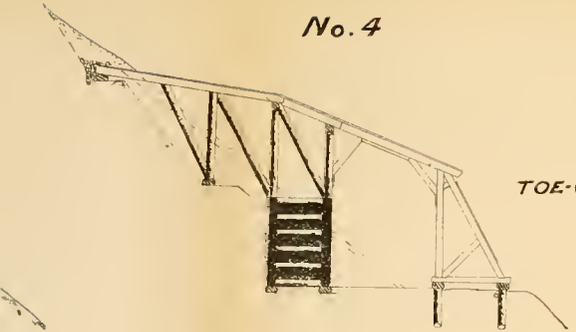
No. 10



LEVEL FALL SHED.

PLATE IX
TRANS AM SOC CIV ENGNS
VOL XIX Nº 394
KEEFER ON CANADIAN
PACIFIC RAILWAY

No. 4



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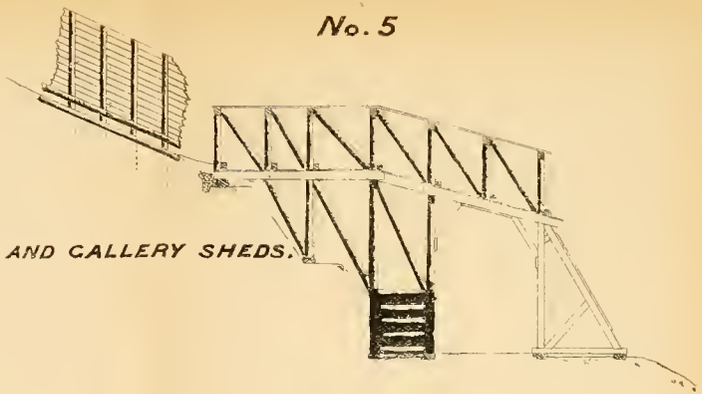
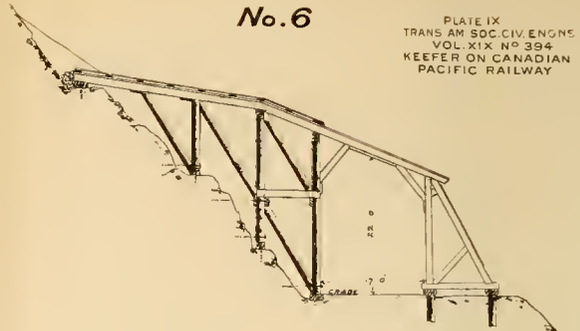


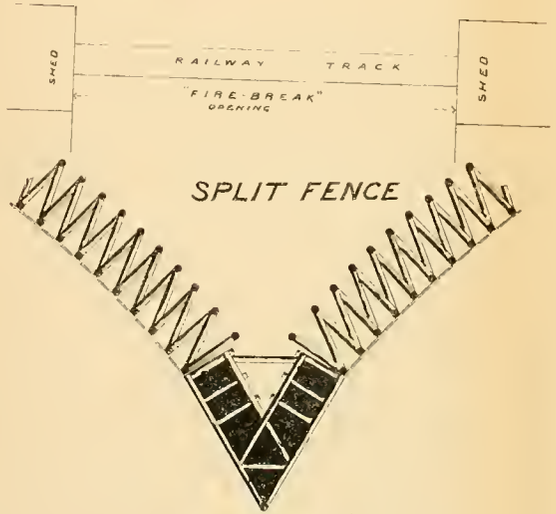
TOE-CRIB AND GALLERY SHEDS.

SCOOP SHED

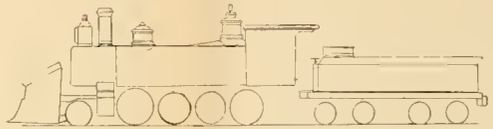
No. 6



GALLERY SHED.

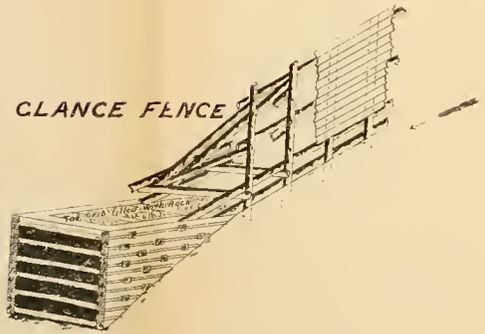


SPLIT FENCE



OUTLINE OF CONSOLIDATION ENGINE AND TENDER.

CLANCE FENCE



C. P. Ry.

SNOW SHEDS.

SCALE 20 Ft. = 1 INCH.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

395.

(Vol. XIX.—September, 1888.)

SOME FACTS IN RELATION TO FRICTION, WASTE AND LOSS OF WATER IN MAINS.

By CHARLES B. BRUSH, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE 28TH, 1888.

WITH DISCUSSION.

It is well known that many engineers have valuable data which they hesitate to publish because they feel that their experiments are not complete. As a matter of fact, these experiments are seldom completed during one's lifetime, and if the information is not properly tabulated and made known while it is fresh in the mind of the investigator, it is apt to be laid aside and lost. Again, the results obtained are frequently unsatisfactory and do not seem to come up to the standard which is generally expected. An engineer often hesitates about making statements which seem to reflect on the operations of the plant under his control, or upon his own efficiency. In consequence of lack of knowledge as to practical results we have erroneous calculations and false expectations. This paper has been prepared largely with the hope of drawing out the experience of others.

Friction, or Head in Feet necessary to obtain different Velocities in Pipes of given Diameters.

On November 15th, 1881, the City of Hoboken contracted with the Hackensack Water Company, reorganized, for a new water supply. The plant was ready November 1st, 1882, and since that time it has been in operation. This plant supplied Hoboken, an old unmetered district, for about one year, but after that time it has in addition furnished water to new districts, which never before had a public supply and in which every service pipe is metered. Since 1882 the total of the population supplied in all the districts has been increased from 33 000 to 75 000—the total taps from 2 700 to 5 600—the total meters from 47 to 2 667—the total length of cast-iron main from 16 to 77 miles—while the total amount of water pumped has only increased from 4 000 000 to 4 250 000 United States gallons per day.

For five years the water was pumped to a reservoir through a single line of 20-inch cast-iron pipe, 75 000 feet long, under a static head of 165 feet. The main is laid along railroads and highways with a large number of summits, angles and curves. No special attempt was made to obtain true lines and grades. The exigencies of a public water supply have naturally produced irregularities in our results, but no attempt has been made to harmonize apparent incongruities. The average delivery has been about 4 000 000 gallons per day, but the meter records show that of this amount about 200 000 gallons per day are taken from the main before the balance of the water reaches the reservoir; 100 000 gallons being taken at a point about 13 500 feet from the pumping station and the remaining 100 000 gallons is taken at a point about 70 000 feet from the station. This 200 000 gallons is not leakage; it is water delivered to consumers located at about these points. The main is laid partly in sandy soil, partly through rock cut, and the balance through soft marsh lands. It crosses numerous culverts and small streams, beside passing under two rivers 400 feet wide and from 15 to 25 feet deep. The elevation of the main above mean high tide for the first 34 000 feet is from 2 to 18 feet—at 37 000 feet from the pumping station its elevation is 35 feet—between 36 000 and 48 000 feet from said station the elevation is from 3 to 6 feet, and for the balance of the line the following are the principal elevations:

At 50 000	feet	from the	pumping station	the elevation is 46 feet.
" 52 000	"	"	"	13 "
" 53 000	"	"	"	28 "
" 55 000	"	"	"	20 "
" 60 000	"	"	"	74 "
" 64 500	"	"	"	8 "
" 66 000	"	"	"	29 "
" 69 500	"	"	"	17 "
" 71 000	"	"	"	166 "
" 71 800	"	"	"	157 "
" 73 000	"	"	"	172 "
" 74 000	"	"	"	148 "
" 75 000	"	"	"	180 "

Among the horizontal bends in the pipe there may be mentioned 4 right angles and 10 quadrants of about 30 feet radius.

The friction head on this main has been found to be considerably less than most of the formulas of standard authorities would indicate.

For the purpose of comparison I have tabulated, after allowing 5 per cent. for slip of pumps, the results actually obtained and those obtained from given formulas, using the same data as to head, length and diameter in each case.

This table indicates that the amount of water that can be most economically delivered through a 20-inch main under a static head of 165 feet is about 3 350 000 gallons per day; a delivery which necessitates a velocity 2.36 feet per second and develop a friction head of 1.029 feet per 1 000 feet, or 5.43 feet per mile. Under these conditions the cost of the coal consumed was $8 per million gallons pumped.

With the same pumps (two compound condensing Duplex Low Duty three million gallons Worthington engines) it cost $8.40 and $9 60 per million gallons to pump respectively 2 848 000 and 4 255 200 gallons per day, through the same main.

The increased cost on the lower velocities was probably due to the fact that the plant could not be operated to its greatest efficiency. The increased cost on the higher velocities was probably due to increased friction.

The formulas which agree best with the results we have obtained are those of Lampe and D'Arcy.

RESULTS OBTAINED BY THE HACKENSACK WATER COMPANY, REORGANIZED, FROM 1882–1887 IN PUMPING WATER THROUGH A 20-INCH CAST-IRON MAIN, 75 000 FEET LONG, UNDER CONDITIONS, ABOVE STATED. ALSO RESULTS WITH SAME DATA OBTAINED FROM STANDARD FORMULAS, AS TO FRICTION, VELOCITY AND DISCHARGE.

Table No. 1, *showing actual results obtained by the Hackensack Water Company, Reorganized.*

Pressure in lbs. per sq. in. at Pumping Station	95	100	105	110	115	120	125	130
Total effective head in feet	55	66	77	89	100	112	123	135
Discharge in United States gallons in 24 hours	2 848 000	3 165 200	3 354 250	3 566 800	3 804 400	3 904 200	4 116 750	4 255 200
Actual velocity in main in feet per second	2.00	2.24	2.36	2.52	2.68	2.76	2.92	3.00
Cost of coal consumed in delivering each million United States gallons at given velocities	$8 40	$8 15	$8 00	$8 10	$8 30	$8 60	$9 00	$9 60

Table No. 2, *showing actual and theoretical friction heads per* 1 000 *feet at velocities given in Table No.* 1, *ranging from* 2 *to* 3 *feet per second.*

Hackensack Water Co., Reorganized	0.729	0.876	1.029	1.186	1.337	1.490	1.645	1.797
D'Arcy. $h = .020889 \frac{lv^2}{2\,dg}$	0.779	0.977	1.083	1.236	1.397	1.482	1.659	1.752
Weisbach. $h = \left\{ 0.1439 + \frac{.017155}{v^{\frac{1}{2}}} \right\} \frac{lv^2}{2dg}$	0.998	1.222	1.343	1.505	1.680	1.772	1.963	2.061
Tables published by the Cincinnati and Newport Pipe Co.	0.928	1.146	1.266	1.438	1.615	1.705	1.904	2.008
Prony. $v = 97.05\ (rs)^{\frac{1}{2}}$	1.136	1.371	1.516	1.722	1.940	2.055	2.293	2.417
Kirkwood. $h = \frac{v^2\,l}{1600\,d}$ For tuberculated pipe	1.499	1.880	2.088	2.380	2.692	2.856	3.196	3.374

Table No. 3, showing actual and theoretical velocities in feet per second due to heads given in Table No. 1, ranging from 55 to 135 feet.

Hackensack Water Co., Reorganized	2.00	2.24	2.36	2.52	2.68	2.76	2.92	3.00
Lampe. $v = 203.3\ r^{.694}\ s^{.555}$	2.015	2.231	2.429	2.632	2.809	2.980	3.151	3.318
D'Arcy. $v = 111.00\ (r\,s)^{\frac{1}{2}}$	1.941	2.125	2.295	2.468	2.617	2.770	2.902	3.040
Chezy. $v = 100\ c\ (r\,s)^{\frac{1}{2}}$								
Leslie. $v = 100\ (rs)^{\frac{1}{2}}$	1.747	1.914	2.067	2.201	2.356	2.494	2.648	2.736
Beardmore. $v = 50\left(\frac{hd}{l}\right)^{\frac{1}{2}}$								
Prony. $v = 97.05\ (rs)^{\frac{1}{2}}$	1.695	1.857	2.006	2.146	2.286	2.425	2.569	2.655
Eytelwein. $v = 47.8\left(\frac{hd}{l + 54\,d}\right)^{\frac{1}{2}}$	1.672	1.832	1.978	2.127	2.255	2.386	2.501	2.620
D'Aubuisson. $v = 95.6\,(r\,s)^{\frac{1}{2}}$	1.670	1.839	1.976	2.104	2.252	2.384	2.531	2.616
Blackwell. $v = 95.8\ (r\,s)^{\frac{1}{2}}$. Accounting for curves	1.674	1.834	1.980	2.109	2.257	2.389	2.537	2.621

Table No. 4, showing actual and theoretical discharge, in United States gallons, in 24 hours, due to heads given in Table No. 1, ranging from 55 to 135 feet.

Hackensack Water Co., Reorganized	2 848 000	3 165 200	3 354 250	3 566 800	3 804 400	3 904 200	4 116 750	4 255 200
Lampe	2 848 550	3 153 930	3 433 820	3 720 800	3 971 025	4 212 760	4 454 500	4 690 590
D'Arcy	2 743 950	3 004 060	3 244 390	3 488 960	3 699 600	3 915 900	4 102 490	4 297 580
Chezy								
Leslie	2 469 700	2 705 775	2 922 070	3 111 500	3 330 625	3 525 700	3 743 420	3 867 815
Beardmore								
Prony	2 396 000	2 624 550	2 835 000	3 033 550	3 230 950	3 427 300	3 659 250	3 745 450
Eytelwein	2 363 670	2 589 850	2 796 260	3 006 890	3 187 840	3 373 030	3 535 600	3 703 830
D'Aubuisson	2 360 800	2 599 800	2 793 400	2 974 400	3 183 600	3 370 200	3 578 060	3 698 200
Blackwell	2 366 500	2 592 700	2 799 100	2 981 400	3 190 700	3 377 300	3 586 500	3 705 200

Note.
- h = head in feet.
- v = velocity in feet per second.
- r = hydraulic mean radius = $\frac{1}{4}d$.
- s = sine of the inclination.
- l = length in feet.
- d = diameter in feet.
- $g = 32.2$.

D'Arcy's formula, given above, is special for 20-inch pipe. See Paper of Mr. Ellis, read before the American Water Works Association, May 16th, 1883.

Lampe's experiments (Danzig, 1869–71) were made with a 16-inch main on a length of 38 000 feet.

D'Arcy's experiments (Paris, 1849–51) were made with pipes 367 feet long, the largest diameter used being 20 inches.

Nearly all of the other formulas are based on experiments made on pipes of short lengths and comparatively small diameters. The earliest of the formulas given is that of Chezy in 1775. His form, given above, has been followed by most of the later authorities except Weisbach; the principal difference being in their substitution of different constants depending upon the diameter and inclination. Weisbach's constants depend not only upon these, but also upon the velocity. The formulas of Chezy, Leslie and Beardmore are practically the same. The formulas of Prony, Eytelwein, D'Aubuisson and Blackwell agree very closely with each other.

For further particulars see "Hydraulics," by Hamilton Smith, Jr., M. Am. Soc. C. E., 1886.

The mains upon which Lampe and D'Arcy made their experiments were laid with extraordinary care and with a view to these experiments.

Our actual results given in these tables are especially interesting, in view of the fact that only ordinary precautions were taken and no attempt was made to avoid angles, high intermediate summits and numerous sharp curves.

Waste, and Methods of its Prevention, with Special Reference to Metering and Minimum Rates.

On November 1st, 1882, Hoboken had a population of about 33 000 people. About 16 miles of water main had been laid up to that time by the city. For twenty-five years previous Hoboken had been supplied from the Jersey City Water Works, practically under the house-rate system. For the first six months of the first year of the new supply the average daily consumption was nearly four million gallons per day.

I was convinced that one-half of this draft was simply waste, useless and injurious to consumers and disastrous to the Water Company. Prompt action was taken, and the result has been that without inconvenience to the public, without restricting any legitimate use of water, and almost without the knowledge of our consumers, we have reduced this waste to the extent of at least one and-a-half million gallons per day. The waste from the mains and the waste of consumers was about equal, and in each case it has been reduced three-fourths of a million gallons per day.

The following table shows a comparison of the results obtained during the first six months of the first and fifth years of the new supply.

	Six months from November 1st, 1882, to May 1st, 1883.	Six months from November 1st, 1887, to May 1st, 1888.
Population	33 000	41 000
Taps, Unmetered	2 667	2 648
Taps Metered	47	822
Total Taps	2 714	3 470
Daily Draft, United States Gallons:		
Unmetered	3 471 645	2 022 843
Metered	459 099	945 975
Total	3 930 744	2 968 818

It is true that this result is not entirely due to the attachment of meters. Other precautions were also taken. Twice each year house to house inspections were made, as well as night inspections of the mains and service pipes. Nevertheless, the reduction of waste in service pipes was mainly due to metering such consumers as were supposed to be most wasteful. We have found it not only the most effectual method

of restricting waste of consumers, but, as will be hereinafter shown, we have also found that universal metering materially assists in determining leakage from mains.

Since this waste has been checked we have been able to maintain a pressure on the city 25 per cent. higher than was practicable while the waste existed. This has added materially to the efficiency of the fire service, domestic convenience, etc., and thus is of vital interest to the city.

The term "minimum rate," as used in this paper, refers only to metered consumers. It is the least annual amount of money that the Water Company will accept from any consumer. This amount must be paid, even if the quantity of water used during the year does not equal in value this minimum rate. Any quantity of water used in excess of the quantity paid for by the minimum rate, is charged to the consumer at regular schedule prices.

The establishment of a minimum rate is desirable for sanitary reasons, as it prevents a parsimonious use of water. It is usually fixed somewhat below ordinary house rates.

This paper compares the revenue derived from consumers in cases where no minimum rates were in force.

There is no minimum rate in Hoboken, and in a well built-up district, accustomed to a public water supply, the same necessity does not seem to exist for minimum rates as in a locality that has never had such a supply. Even in new localities the amount of water used increases as consumers become better acquainted with the advantages of a public supply, because they find it more economical to obtain water from that source than from existing wells. Nevertheless, in our new districts, after three years' trial without it, we were obliged to establish a low minimum rate.

The diagram, Plate X, annexed to this paper, showing the draft each hour after the waste of Hoboken had been reduced about 32 gallons per head each day, is very suggestive. The draft between 22 and 4 o'clock, even after the reduction, was still at the rate of $2\frac{1}{3}$ millions per day, or 69 gallons per capita.

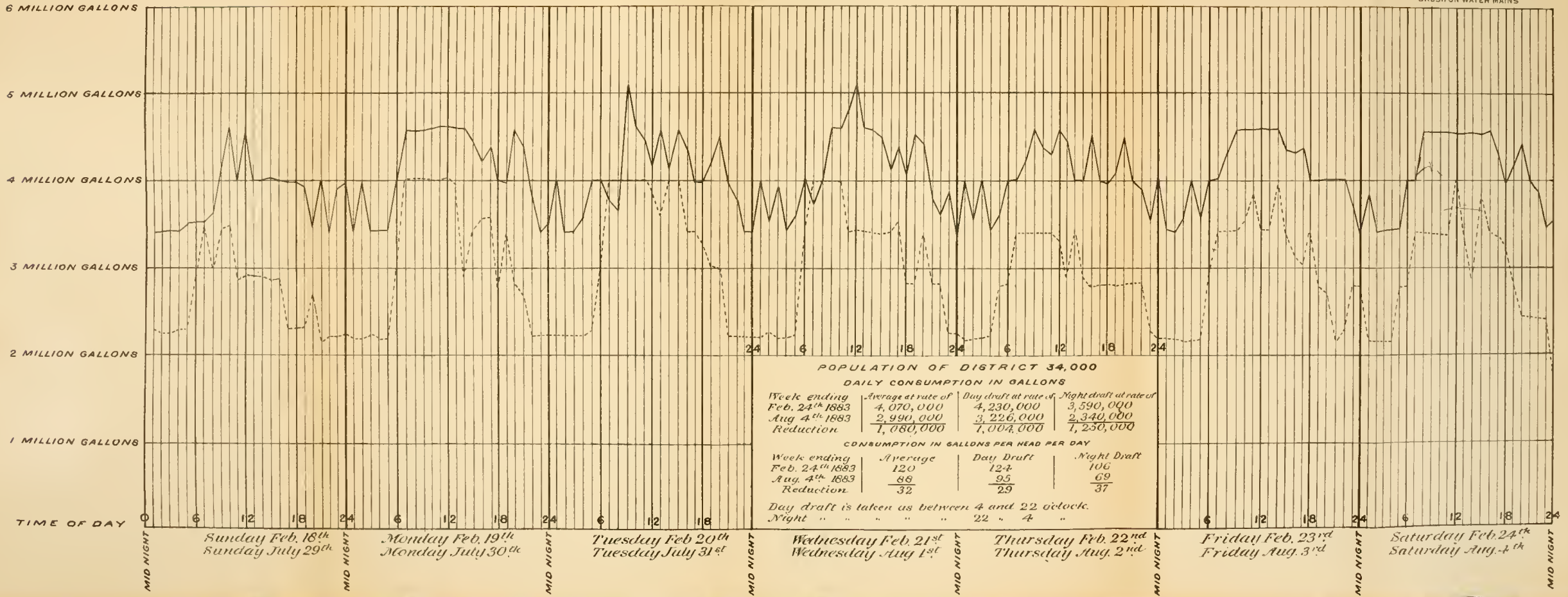
Where does this water go ?

The legitimate consumption can only be a small percentage of this amount. Since 1883, when this diagram was prepared, the waste of Hoboken has been still further reduced, but the arrangement of our plant is such that it is now impracticable to obtain an accurate record of the hourly draft of this district.

PLATE X
TRANS AM SOC CIV ENGNS
VOL XIX Nº 395.
BRUSH ON WATER MAINS

HACKENSACK WATER CO., REORGANIZED.

Comparison of draft of Hoboken each hour for
Week ending Feb. 24th 1883 shown by full irregular line.
" " Aug. 4th " " " broken " "
Between these dates the waste was largely reduced.



POPULATION OF DISTRICT 34,000

DAILY CONSUMPTION IN GALLONS

Week ending	Average at rate of	Day draft at rate of	Night draft at rate of
Feb. 24th 1883	4,070,000	4,230,000	3,590,000
Aug 4th 1883	2,990,000	3,226,000	2,340,000
Reduction	1,080,000	1,004,000	1,250,000

CONSUMPTION IN GALLONS PER HEAD PER DAY

Week ending	Average	Day Draft	Night Draft
Feb. 24th 1883	120	124	106
Aug. 4th 1883	88	95	69
Reduction	32	29	37

Day draft is taken as between 4 and 22 o'clock.
Night " " " " " 22 " 4 "

In order to illustrate the ideas I wish to convey, the following tables are submitted.

The actual effect of the attachment of meters during the fall of 1885 on four hundred and thirty-nine houses in Hoboken, previously supplied for many years on the House Rate System, may be ascertained by an examination of the following table:

Consumers.	Amounts Paid by Consumers. Former House Rates.	Present Meter Rates.	Difference.
November 1st, 1885, to February 1st, 1886.			
83 per cent. reduced	$2 978 39	$2 047 92	$930 47
17 " increased	687 41	1 042 73	355 32
	$3 365 80	$3 090 65	Loss, $575 15
February 1st, 1886, to May 1st, 1886.			
63 per cent. reduced	$2 382 13	$1 433 28	$948 85
37 " increased	1 424 87	2 610 50	1 185 53
	$3 807 00	$4 043 78	Gain, $236 68
May 1st, 1886, to August 1st, 1886.			
59 per cent. reduced	$2 254 88	$1 453 26	$801 62
41 " increased	1 566 95	2 775 17	1 208 22
	$3 821 83	$4 228 43	Gain, $406 60
August 1st, 1886, to November 1st, 1886.			
57 per cent. reduced	$2 188 45	$1 447 16	$741 29
43 " increased	1 633 38	2 813 29	1 179 91
	$3 821 83	$4 260 45	Gain, $438 62
November 1st, 1886, to February 1st, 1887.			
72 per cent. reduced	$2 751 72	$1 941 04	$810 68
28 " increased	1 070 11	1 948 81	878 70
	$3 821 83	$3 889 85	Gain, $68 02

Consumers.	Amounts Paid by Consumers. Former House Rates.	Present Meter Rates.	Difference.
February 1st, 1887, to May 1st, 1887.			
75 per cent. reduced	$2 684 71	$1 606 88	$1 077 83
26 " increased	1 025 35	1 661 09	636 74
	$3 710 66	$3 267 97	Loss, $442 09
May 1st, 1887, to August 1st, 1887.			
58 per cent. reduced	$2 216 67	$1 558 00	$658 67
42 " increased	1 605 16	2 737 72	1 132 56
	$3 821 83	$2 895 72	Gain, $473 89
August 1st, 1887, to November 1st, 1887.			
62 per cent. reduced	$2 431 26	$1 580 34	$850 27
38 " increased	1 405 16	2 450 43	1 045 92
	$3 836 42	$4 030 77	Gain, $194 35
November 1st, 1887, to February 1st, 1888.			
67 per cent. reduced	$2 477 78	$1 709 49	$768 29
33 " increased	1 160 37	1 981 67	821 30
	$3 638 15	$3 691 16	Gain, $53 01
February 1st, 1888, to May 1st, 1888.			
70 per cent. reduced	$2 650 14	$1 947 87	$702 27
30 " increased	1 059 92	1 833 83	773 91
	$3 710 00	$3 781 70	Gain, $71 64

In order to ascertain the effect of the meter system in a new district, one hundred and ninety-one average dwellings were selected, connected with our high service plant, which contained about eight occupants to each house. These houses were metered. If house rates in each case had been charged, the daily draft *per capita* would have been about 75 gallons, and the following result would have been obtained:

Houses.	Annual Revenue.	Daily Draft.
191	$2 645 48	115 000 gallons.

The following have been the actual results:

			Annual Revenue.	Daily Draft.
First	six months,	winter....	$1 081 32	12 860 gallons.
Second	"	summer...	1 617 26	18 948 "
Third	"	winter....	1 510 44	18 000 "
Fourth	"	summer ..	1 859 94	24 000 "
Fifth	"	winter....	1 385 17	16 550 "
Sixth	"	summer ..	1 959 24	23 000 "
Seventh	"	winter....	1 751 52	21 645 "
Eighth	"	summer ..	2 083 84	25 387 "
Ninth	"	winter....	1 825 92	22 516 "

At house rates each house would have averaged $13 85 per annum.
At meter rates the actual return was $9 56.

At rate of	first six months,	winter.....	$5 66	per annum.
"	second "	summer.....	8 46	"
"	third "	winter.....	7 90	"
"	fourth "	summer.....	9 74	"
"	fifth "	winter.....	7 58	"
"	sixth "	summer.....	10 25	"
"	seventh "	winter.....	9 16	"
"	eighth "	summer.....	10 91	"
"	ninth "	winter.....	9 56	"

Each corresponding period shows a constantly increasing use of water.

It will be noticed that in a metered district the draft in winter is less than that in summer. In unmetered districts the reverse is the case. During the extremes of cold and hot weather the draft *per capita* is about the same in both districts, but after the extreme weather has passed, the waste in the metered district immediately ceases, while in the unmetered district the waste continues for some time.

CLASSIFICATION OF CONSUMERS OF THE HIGH SERVICE DISTRICT OF THE HACKENSACK WATER COMPANY, REORGANIZED, ACCORDING TO AMOUNTS PAID ANNUALLY PREVIOUS TO THE ESTABLISHMENT OF A MINIMUM RATE. ALL THE CONSUMERS WERE METERED.

CLASSIFICATION. Amount of Water consumed per annum.	NO. OF CONSUMERS IN EACH CLASS ON QUARTERS ENDING								
	Feb. 1886.	Nov. 1885.	Aug. 1885.	May 1885.	Feb. 1885.	Nov. 1884.	Aug. 1884.	May 1884.	Feb. 1884.
Between $0 00 and $2 00	164	107	57	128	127	66	37	66	66
2 00 " 4 00	192	165	123	125	107	93	60	46	52
4 00 " 6 00	124	106	67	62	71	67	37	30	25
6 00 " 8 00	55	72	65	38	36	40	32	12	20
8 00 " 10 00	43	45	37	32	30	19	22	16	13
10 00 " 12 00	40	20	39	22	15	16	9	10	8
12 00 " 16 00	49	52	36	21	24	31	24	12	9
16 00 " 20 00	26	27	29	17	14	21	9	5	7
20 00 " 40 00	58	55	69	52	32	37	31	10	7
40 00 " 80 00	12	17	10	10	9	11	7	8	2
80 00 " 200 00	7	5	10	10	4	5	5	4	4
Over $200 00.....	5	5	6	5	5	4	4	3	3
Total.....	775	676	548	522	474	410	277	222	216

It will be seen from this table that during the latest quarter three hundred and fifty-six consumers, out of a total of seven hundred and seventy-five, used less than four dollars worth of water per annum, and practically this proportion of small consumers appears in each of the nine quarters given. In order to check this extreme economical tendency of some of our consumers in a new district, the company was obliged, during the year 1886, to establish a low minimum rate depending upon the size of the meter attached to the consumer's service pipe.

The result of the experience of this water company is that the meter system is mutually advantageous both to the consumer and the company. Each metered consumer pays only for the water he uses, and there are no restrictions either as to quantity used or the time of use. No inquisitorial examination of premises, so unpleasant to consumers, are required. It is the interest of each consumer to have good plumbing. Those who are careless and wasteful, and those who require or desire to use more than an average amount of water, must and should pay for such excess, but the balance, constituting two-thirds of all the

consumers, pay less by meter than by house rates. Meters were attached only after intense and prolonged opposition. Common sense prevailed, however, and now they are quite popular.

Our experience to date indicates that the true policy of a Water Board is to furnish meters and keep them in repair free of expense to the consumer, except in case of misuse or freezing. The meters are then entirely under the control of the Board, who will naturally see that repairs are promptly made. The consumer's interest is rather to see that such repairs are delayed.

The following is an extract from a letter I have received from Wm. G. Richards, Superintendent of the Water Works at Atlanta, Georgia:

"Our works are city property, and are used for almost any and all purposes for which the city can use water, such as puddling ditches after laying sewers or water mains, flushing sewers, and frequently for washing gutters, so that anything like a comparison of the pumping with the meter records would be an impossibility. Then, the public schools are not metered, the city claiming the right to use all the water they want.

"Meters were introduced here as an absolute necessity. The pumpage had become so great, that, with our machinery, it was impossible to maintain a satisfactory pressure, and the city was not in a condition to put in more pumping machinery and a larger pumping main. Meters have now been in general use here about 3 years, and the pumpage does not reach an average of 2 000 000, and before their general introduction it frequently reached 5 000 000 gallons per day. Our number of consumers has regularly increased, but the waste is stopped, and the pressure is satisfactory all over the city."

It is unfortunate that the question of metering is almost synonymous in the public mind with injustice and jobbery. Until a community can be educated so as to grasp the importance of the subject, both from a pecuniary and a sanitary standpoint, and until the moral sense of a community can be sufficiently advanced not to tolerate rascality, such a community will have to suffer and retrograde. The public will learn in time, however, that in most localities the amount of desirable water that can be obtained is limited—that even if it can be obtained in abundance the expense of the supply depends directly upon the quantity—and that the danger to the public health, public safety and the public welfare is directly in proportion to the amount of waste.

Loss of Water from Mains.

In November, 1883, the Hackensack Water Company determined, and has thereafter metered every consumer in each new district it supplied. This is the first instance, I believe, where such radical action has ever been taken. Since that time, Princeton has adopted the same policy, but with this exception I do not know of any other locality where every service is actually metered, though there are many places where a large majority of the taps are metered.

In consequence of universal metering, this company is enabled, in certain districts, to ascertain the difference between the amount of water supposed to be delivered into the mains and the amount drawn through the service pipes of consumers.

The balance may be grouped as follows:

A.—Water drawn through fire hydrants for fire purposes, flushing mains and sewers, settling trenches, street sprinkling, etc.

B.—Slip of pump.

C.—Slip of meters.

D.—Clerical errors.

E.—Leakage from mains.

What becomes of this balance is a question which is receiving earnest consideration. All the meters are tested on large and small streams and found correct before they are attached, and when they are taken off for examination or repairs they are tested and are rarely found inaccurate to a greater extent than three per cent.; usually they come considerably within that limit. We have not as yet sufficient data to determine the proportion of the balance which should be credited under each of the above heads, but the total of the actual balance and such data as we have, I submit in each case, and leave it for you to draw your own conclusions.

High Service District.

The amount of water delivered into these mains is measured by the displacement of the high service pumps. The mains are from 4 inches to 16 inches in diameter, and 22.37 miles long. There are one thousand four hundred and fifty-nine consumers from these mains, and one hundred and ninety-five fire hydrants on this system. The pressure ranges from 25 to 75 pounds per square inch.

Quarter Ending.	No. of Consumers.	Daily Delivered to Mains. U. S. Gals.	Daily measured by separate Meters. U. S. Gals.	Daily Balance. U. S. Gals.	Balance per cent.
November 1st, 1883	180	45 076	16 213	28 863	64
February 1st, 1884	227	75 444	33 314	42 130	55
May 1st, 1884	284	88 785	43 939	44 845	51
August 1st, 1884	418	137 306	75 886	61 420	45
November 1st, 1884	487	125 709	77 593	48 116	38
February 1st, 1885	539	107 508	74 038	33 470	31
May 1st, 1885	570	138 638	101 503	37 133	27
August 1st, 1885	702	176 979	130 195	46 784	27
November 1st, 1885	771	179 385	109 629	69 758	39
February 1st, 1886	804	166 256	110 537	55 719	33
May 1st, 1886	830	207 027	126 996	80 031	38
August 1st, 1886	912	266 281	156 606	109 676	41
November 1st, 1886	1 035	321 784	186 745	135 038	42
February 1st, 1887	1 099	311 002	171 051	139 951	45
May 1st, 1887	1 155	324 473	191 439	133 034	41
August 1st, 1887	1 271	460 274	276 164	184 130	40
November 1st, 1887	1 373	452 455	262 424	190 031	42
February 1st, 1888	1 416	729 640	510 748	218 982	30
May 1st, 1888	1 459	552 293	270 624	281 569	51

Englewood Distribution Mains.

The amount of water delivered into these mains is measured by a 6-inch meter. The mains are from 4 to 8 inches in diameter and 4.78 miles long. There are eighty-six consumers and forty fire hydrants on this system. The pressure ranges from 25 to 100 pounds per square inch.

Quarter Ending.	No. of Consumers.	Daily Deliv-ered to Mains.	Daily Gallons Measured by Separate Meters.	Daily Balance.	Balance Per Cent.
February 1st, 1888.....	70	9 709	3 883	5 836	60
May 1st, 1888.....	86	17 041	9 543	7 498	44

Ridgefield Pipes.

The amount of water delivered into these mains is measured by a three-quarter inch meter. The pipes are wrought iron, 2 inches in diameter, 0.40 miles long. There are seventeen consumers on this system and no fire hydrants. The pressure ranges from 50 to 100 pounds per square inch.

Quarter Ending.	No. of Consumers.	Daily Delivered to Mains.	Daily Gallons Measured by Separate Meters.	Daily Balance.	Balauce Per cent.
November 1st, 1885.....	9	1 487	1 322	165	11
February 1st, 1886.....	11	1 514	1 019	495	32
May 1st, 1886.....	11	2 351	985	1 366	58
August 1st, 1886.....	12	2 917	1 616	1 301	44
November 1st, 1886.....	14	2 325	2 266	59	2
February 1st, 1887.....	17	1 647	1 515	82	4
May 1st, 1887.....	17	1 695	1 534	161	8
August 1st, 1887.....	17	3 672	3 348	324	9
November 1st, 1887.....	17	3 037	2 929	108	4
February 1st, 1888.....	17	3 301	2 968	333	11
May 1st, 1888.....	17	2 858	2 497	361	14

In addition to the above data Prof. Libbey, of the Princeton Water Works, informs me that they have 6 miles of main on which this balance is 20 per cent., or 20 000 gallons per day. He thinks, however, that the loss is principally due to a leak in the main, which he has not yet been able to find, but which he believes he has located within a length of 4 500 feet on a certain main.

Your attention has already been called to the fact that on the 16 miles and more of mains in Hoboken, leakage to the extent of 750 000 gallons per day has actually been checked.

In this connection I also submit an experiment made to determine the leakage on a 24-inch cast-iron force main laid last year by the Hackensack Water Company. The main is 11 miles long, and at the time of the test it was supplied with gates, air-cocks and blow-offs, but no hydrants. The main had been carefully examined under heavy pressure for several weeks after the work was completed, and there was no evidence whatever of leakage. It was connected at both ends with the existing 20-inch main. About midway, the 24-inch main was connected with an iron tank at the summit of the whole system. The test was made by opening the gates at each end of the 24-inch main and filling the tank

under a pressure of 107 pounds per square inch. The gates were then closed and the pressure in the 20-inch main raised to 110 pounds per square inch. The loss on this 24-inch main, 11 miles long, was then found by tank measurement after several trials to be about 70 000 gallons per day. The main was composed of excellent casting, with good lead joints, well caulked and laid by experienced men under careful inspection, by day's work. One thing, at least, is true, viz., that serious leaks may exist in a main under heavy pressure and yet not appear on the surface of the ground.

Perhaps the greatest folly of our time is the almost universal attempt of cities and towns to increase their water supply plant to keep pace with their waste. It is a hopeless task. It amounts to the same thing as attempting to fill a pail which has only a sieve for a bottom. The amount of water that can be used is limited; the amount that can be wasted has no limit. The patience of man has a very positive limit, and therefore I do not consider as waste any amount of water drawn from a self-closing cock held open by hand—drawn for sprinkling gardens, lawns, roads, sidewalks and buildings through a hose held in the hand—drawn for fountains or for use in the various forms of business—but I do insist that water leaking from street mains, service pipes and fixtures out of order, is useful to none, a damage to the individual consumer, and a serious tax on the community. Flushing pipes and drains from time to time is not waste; but allowing small streams of water to run continuously from partially open faucets and cheap water closets is waste of the most serious character, and if not continually watched and checked will soon reach alarming proportions. Where water has to be pumped I do not believe that its use as a motive power is legitimate, except for small eccentric service.

In conclusion, I take pleasure in acknowledging the assistance rendered to me in the preparation of tables given in this paper by L. L. Tribus, Jun. Am. Soc. C. E.

DISCUSSION.

G. H. Benzenberg, M. Am. Soc. C. E.—I wish to express my personal thanks to Mr. Brush for the presentation of his paper at this time. It refers to a question which has been agitating our citizens here for a year or two past. Our experience with reference to the waste of water has been largely similar to that of Mr. Brush. The consumption per capita here has been from 110 to 115 gallons per day, and rapidly increasing beyond that figure, until last summer it had reached the amount of 128 to 130 gallons per day.

Thereupon, taking advantage of a law which had been passed during the previous winter, meters were largely forced by the department on to the water service pipes, supplying business places and manufactories, and the result was an almost immediate reduction in the demand upon the supply of water, which continued during the entire fall and early winter, until the diminution amounted to almost eight million gallons a day, or a reduction equal to from 38 to 40 per cent. of the present consumption.

During the heaviest consumption last summer the demand had reached to within 5 per cent. of the total pumping capacity and to within 2 per cent. of the capacity of the intake, and the department was without any means or authority to increase either at once, so that the result of placing these meters, materially reducing the waste and increasing the capacity in reserve at the pumping works plant from 2 to 38 per cent., was gratefully appreciated. It was further observed from the record of the consumption taken hourly that the proportionate reduction had been much more during the hours of 11 in the evening to 4 o'clock in the morning, than during any other hours of the day, showing that the reduction had mainly consisted of the former waste. Heretofore the consumption during those hours was at the rate of from 70 to 82 gallons per day to every inhabitant, man, woman and child, in the city, or about 70 per cent. of the average hourly consumption, whereas since last fall the consumption during those same hours is at the rate of about 50 gallons per capita per day, or 60 per cent. of the average hourly consumption, which must, however, still be considered largely as waste.

The number of meters that were connected during the time that this reduction took place was about 1 000, and I mention these facts only as additional proof to what Mr. Brush has stated, that metering is "the most effectual method of restricting waste of consumers."

J. B. Francis, Past President Am. Soc. C. E.—There is one point that I want to ask Mr. Brush about, and that is the condition of the interior of the pipe; the loss due to friction depends very much on the

condition of the interior of the pipe. I have not heard anything about the matter of coating pipes to keep them from oxidation. What means were used, and with what result? If the condition of the interior of the pipe was stated, it would add to the value of the paper.

Mr. Brush.—The pipes were new, of cast-iron and coated in the usual way with tar. They have been in use only five years. There was no oxidation on the pipes.

Mr. John W. Weston.—I would like to ask Mr. Brush if he has experienced any trouble from "wind" in the distribution pipes as we have in Chicago. For several years past the daily papers have had a fund of matter on hand for emergencies, in the way of complaints against the water department on account of insufficient supply and failure in head to reach the upper floors of the ordinary down-town business buildings. An entire change has come about since the advent of the new city administration and the appointment of Mr. F. W. Gerecke, of Newburgh, N. Y., as Chief Engineer.

Immediately on his arrival at the hotel in Chicago, on drawing water Mr. Gerecke noticed the unusual noise and apparent force of the discharge from the faucet, and raising a glass of water to the light observed a host of air-bubbles rising to the surface. This set him to thinking, which he continued, while making close examinations of the pumping machinery. He found that the valves of certain of the pumps were of faulty construction, so that they would not seat themselves quickly enough to hold all the water. To remedy this, previous officials resorted to the introduction of more air than was allowed in the original construction of the pumps. This was accomplished by regular air-compressors, which were obtained and placed in connection with the pump barrels in the basement of the building. A large amount of air was thus forced into the mains.

Of course the immediate effect was to make the pump valves and pumps work easier, and this would seem to have been the only object considered, as for this purpose air compressors were supplied to both pump houses, and a large amount of air regularly introduced.

Upon taking indicator diagrams from the engines the actual result was somewhat startling; for whereas some 850 horse-power should be required for the work done, the cards only showed 614 as the power actually exerted by that particular engine, demonstrating that nearly a third of the contents of the discharge mains was air.

Since February last, examinations and the necessary overhauling and repairs have been quietly going on, with the results that the pumping machinery has been able to do its work without the air compressor attachments, and that the water supply now reaches the fifth stories of the business buildings of Chicago, whereas for years, practically, it was

impossible to obtain water from the public supply higher than the third.

The discovery of this anomalous condition of a large city's water supply, and the simple changes that effected relief and improved its efficiency, are both curious and interesting.

Mr. BRUSH.—The Hackensack Water Company pumps air into the water for purposes of aeration, but the larger portion of this air in the mains is removed through valves at the summits. The balance of the air, that is not absorbed by the water, passes into our reservoirs. We find this aeration of the water very desirable. We have had no trouble with the air interfering with the flow of water in our mains.

T. J. WHITMAN, M. Am. Soc. C. E.—Has Mr. Brush made any estimates of the loss occasioned by the lead joints of his pipe ?

Mr. BRUSH.—I have not. I do not consider that the serious question as to loss in mains is that of waste at each joint. Many lead joints under heavy pressure show moisture, which indicates small seepage, but this does not amount to much in the aggregate. I refer rather to broken or defective pipes, partially blown out joints, leaky taps, connections and fixtures. When the excavation is partly in earth and partly in rock, in a gravelly or in a moist soil, the water from a leak, instead of coming to the surface often runs along the pipe for a considerable distance, and then disappears without showing at all upon the surface of the ground. We have found several 4-inch connections to fire hydrants broken and the water running off through the soil into the sewers. One of these leaks amounted to 300 000 gallons per day, and it had evidently been running for a long time. These leaks, among others, were discovered during our night inspection of the mains. These examinations are made between midnight and 4 A. M. After a little experience a leak can be quite accurately located by listening to the sound of the water passing through partially closed valves on the mains and service pipes.

The value of the tables on pages 103 and 104 largely consists in the fact that we are forced to admit that leakage and waste exists to a great extent, and are thus driven to the necessity of greater vigilance and more careful examinations. This is our own case at present, and I am confident that we will soon be able to locate this waste and stop it.

Mr. WHITMAN.—From 1871 to 1887, in furnishing water to St. Louis, an hourly statement was kept of the flow of water into the distribution system. We found the same trouble that Mr. Brush has found, viz., a large consumption at night. During the last four years we have taken up about 20 miles of distribution mains located in various parts of the city, and I do not think we found any leaks at the joints. Of course we

have breaks, but the point I want to make is this, that I think the joint we use is the most perfect thing that can be obtained, considering its cost and facility of making, and that leaks of any amount are not due to the joints. If a cap comes off of course there is a large leak, but as I read the paper, it seems to indicate that there were leaks in the mains which would go to show that the plan of the joint was imperfect. I want to make my protest against that view.

Mr. Brush.—I agree with Mr. Whitman that a well-made lead joint is excellent and entirely practicable. I refer, however, to special cases of leakage and had no intention of criticising the principle of the lead joint.

J. F. Crowell, M. Am. Soc. C. E.—I would like to ask if the results through one season show the same equality in the minimum consumption as is here shown for a limited time? Would not these data indicate accidental quantities of the discharge? Suppose at the time of the minimum consumption the pressure is as great as at any other time (the pumping is as great) that would thus indicate a constant leakage.

Mr. Brush.—The draft was from a reservoir, not from direct pumping. The pressure therefore increased as the consumption decreased.

(Pointing to the (lower) night draft line on the diagram, see Plate X). This straight line indicates that the house waste has been reduced. It is evident that consumers are not allowing water to run at irregular hours in their houses. Any service, running water continually, however, would not have changed the horizontality of this line. We had stopped irregular waste, but the regular waste still continued.

C. F. Loweth, M. Am. Soc. C. E.—How does Mr. Brush account for the great variations which exist in the last column of the tables on page 13, running from 64 per cent. to 27 per cent.

Mr. Brush.—The large variation in the first two quarters of the table to which you refer is due principally to the blowing off of the new main and connections. There is a large amount of water always wasted during the first few quarters of a new supply. You will notice that for the quarter ending May 1st, 1886, the loss was about 80 000 gallons per day, and that this was the greatest loss that had occurred in any quarter during the first three years of the new supply. After this time, an extension was made about five miles in length through a rocky country. This extension is under heavy pressure, and though no leaks appear on the surface I am satisfied that they exist. The value of this table is not

so much in the last column of the percentages as in the preceding column, showing the number of gallons unaccounted for each day. The former is merely relative, depending upon the draft, the latter is positive and is very valuable information.

CHARLES E. GOAD, M. Am. Soc. C. E.—Have you any record of the quantity of water used in fires? Some water works have a method of getting at this. They have a means of knowing when a fire starts and when it has stopped. Then they estimate how much water has been used.

Mr. BRUSH.—The amount of water thus used is comparatively small. It is less than 1 per cent. of our total consumption. We have a large territory to supply, consisting of many municipalities in which small fires frequently occur. Our relations with the fire companies are not as yet well defined, and therefore I am not able at present to control their records. We try to keep the record, but have not been able to do so accurately. I have found, however, that the amount of water used in putting out an ordinary single house fire ranges from 100 000 to 150 000 gallons.

Mr. GOAD.—Have you no record of a large fire?

Mr. BRUSH.—I am very glad to say that I have not.

Mr. BENZENBERG.—If permitted, I might answer that question by giving one experience we had here. The largest fire we have had was the burning of the Newhall House, at which time every fire engine in the city was drawing water for nearly twenty-four hours. The amount of water used was about sixteen per cent. of the general consumption for the day, but there was almost a continuous use for nearly twenty-four hours.

Mr. BRUSH.—About what is the general consumption?

Mr. BENZENBERG.—About 18 500 000 to 19 000 000 gallons.

Mr. GOAD.—How many companies were there?

Mr. BENZENBERG.—There were at that time seven steamers supplying twelve lines of hose.

Mr. GOAD.—The pressure was about 40 pounds?

Mr. BENZENBERG.—About 45; the static pressure is about 50 in that district.

J. T. FANNING, M. Am. Soc. C. E.—Mr. Brush's paper has suggested interesting lines of thought. I will add a word respecting the valuable Table of Experimental and Theoretical Data relating to the flow of water in his 20-inch diameter main. I am glad that we have the results of his investigations of formulas in this compact form, and especially that he has thus given the results of careful experiments.

Our members ought to be stimulated by this paper to give to the Society the results of their accurate experiments, even though they cover but limited range, so that they may be used in comparison with other similar data and in scientific deductions.

Often, in our practice, we are confronted with new problems, which we can not solve with certainty without the aid of some experimental data.

I recall perplexities in my own early work connected with the use of formulas for flow of water in pipes, as then given in the hand books of the day.

In planning, twenty-five years ago, the water supply for a small city, I was much puzzled in attempting to determine the quantity of water that would be delivered by gravity, through a supply main, to the city, from a gathering reservoir seven miles distant.

Whether the pipe might be 12 inches diameter or must be 16 inches diameter, was so important, that it would decide the question in favor or against the construction of the works, for the largest main would cost 50 per cent. more than the smaller, and the total cost of the works would be increased thereby to an amount that would be considered financially prohibitive.

I compared the results given by the various formulas for total volume of flow under the given total head, and for the amount of head consumed by given lesser volumes of flow, and found extreme discrepancies for like data, which lead to an exasperating uncertainty.

The formulas of Chezy, Leslie, Beardmore, Prony, D'Aubuisson and Blackwell, which are given by Mr. Brush, were then accessible, and gave tolerably agreeing results for the given case, but they gave results differing from the formulas of Du Buat and Weisbach, and on testing with varied diameters and velocities of flow were so discordant as to leave much doubt about which was most trustworthy, or in fact if either might be relied upon in a case of so much importance. I had not at that early date the advantage of Kutter's investigations, and was forced to gather for the purpose of testing these formulas all the experimental results of velocities and volumes of flow in pipes that I could obtain and to compute their co-efficients.*

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* This necessitated the tracing of the origin of the co-efficient as it appears in the formula $v = 100\ (ri)^{\frac{1}{2}}$. The 100 includes the constant factor $\sqrt{2g}$ and the reciprocal of an experimental or variable co-efficient whose symbol is m, whence the co-efficient $100 = \sqrt{\frac{2g}{m}}$. Using these terms, it is evident that the formula for v, in long pipes, may be written in the following equivalent forms, $v = c\ (r\ i)^{\frac{1}{2}} = 100\ (r\ i)^{\frac{1}{2}} = \sqrt{\frac{2\,g\,ri}{m}} = \sqrt{\frac{2\,gd\,n}{4\,ml}}$ from which we have $m = \frac{2\,g\,ri}{v^2} = \frac{2\,g\,d\,h}{4\,7\,v^2}$

As the reduction of experimental co-efficients proceeded, it became evident that the prefixed co-efficient of the formula was not even approximately a constant, but that it varied with diameter of pipe and with velocity of flow.

Plotting the experimental co-efficients of one size of pipe for different rates of flow, using velocities for ordinates and co-efficients for abscisses, and then similarly plotting for other sizes of pipes, I found the plotted points of co-efficients for each size of pipe to lie very nearly in lines of parabolic curvature, and for the different sizes of pipes nearly concentric, but curving more rapidly and tending toward a common point as velocity became small, and tending to parallelism as velocity became excessive.

On readjusting the curves in accordance with the apparent law of increase and decrease of co-efficients, I was then able to fill in curves for the intermediate sizes of pipes not covered by experiments, and to continue the curves slightly beyond the range of experiment, so as to cover the range of ordinary uses of pipes under pressure in water supply systems.*

The diagram indicates that the value of c, which Mr. Brush gives here in Table No. 3 in the Chezy and Leslie formulas, had a range in ordinary city pipe systems and with medium velocities of flow, from about 90 to 135, and that the experience co-efficient embodied in this c had a range from about .0034 to .0074, while c, when of precisely the value 100, corresponds to the experimental co-efficient .00644; also that in clean service pipes the c might have a value as low as 70.

With the aid of the diagram I saw that the first difficulty with the use of the formulas was that I had previously lacked properly classified knowledge of the conditions of flow in pipes to adapt the variables of the formulas, which were given as constants, to the particular case in hand.

With the separation of the formulas into their several factors, and tracing the ranges of the variables in the diagram, there came a revelation of wide significance as to the flow of water under pressure. It appeared that all those formulas, similar to the last seven of the nine formulas given here in Mr. Brush's Table No. 3, are practically identical, and that so long as their co-efficient c, or 100, is treated as constant, the formula applies to only one condition of rate of flow in a given pipe, and to one other rate of flow only in any other size of pipe, and thus if I applied them to my 12-inch pipe they were adapted to a velocity of about $\frac{6}{10}$ths-foot per second, and if to the 16-inch pipe, to a velocity of about $\frac{1}{10}$th-foot per second. These velocities were far below those that must necessarily be used in the case then being considered.

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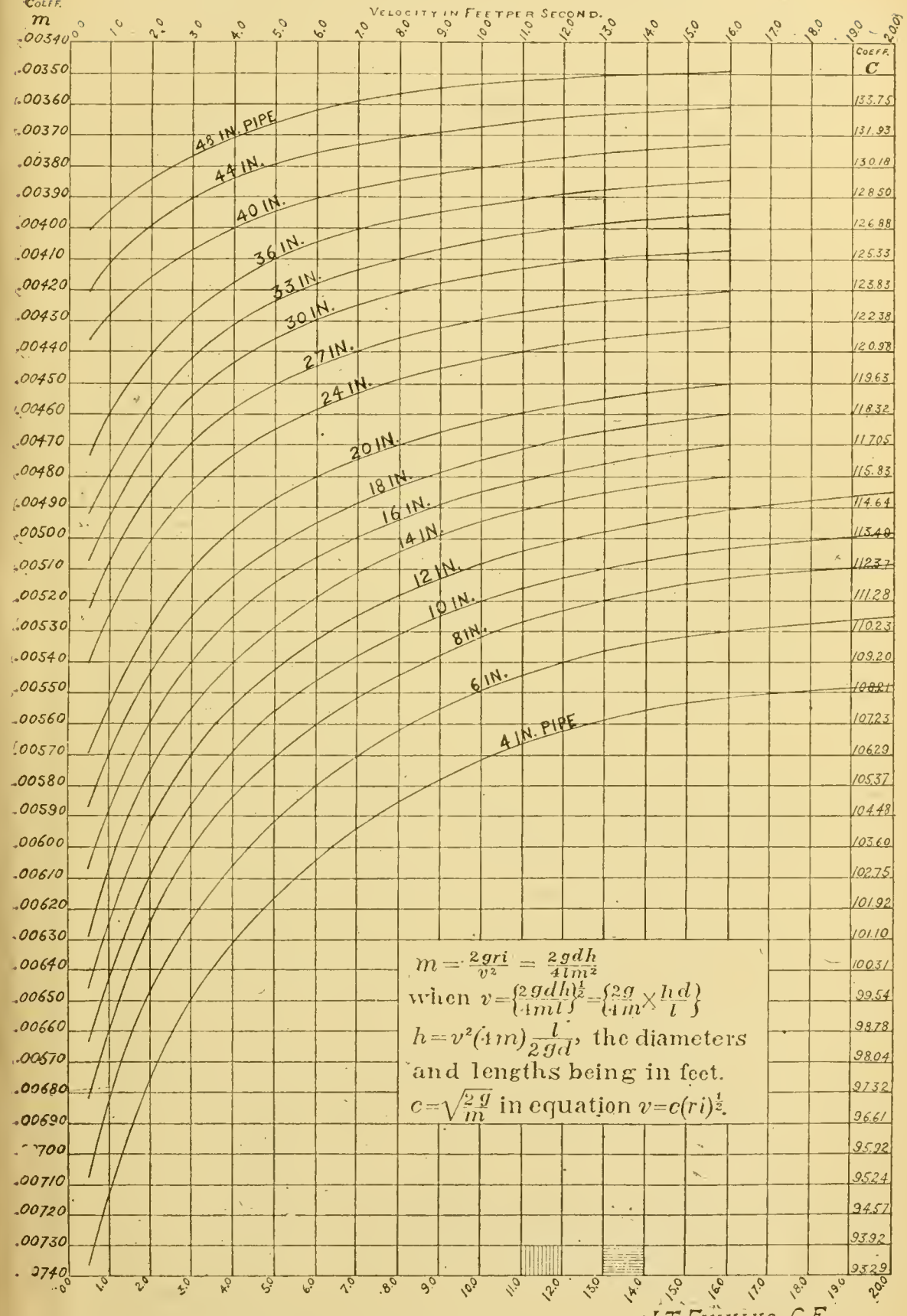
* A reduced copy of the diagram alluded to is presented herewith, Plate XI.

DIAGRAM OF
COEFFICIENTS OF FLOW (m) IN PIPES UNDER PRESSURE.



$m = \frac{2gri}{v^2} = \frac{2gdh}{4lm^2}$

when $v = \left\{\frac{2gdh}{4ml}\right\}^{\frac{1}{2}} = \left\{\frac{2g}{4m} \times \frac{hd}{l}\right\}$

$h = v^2(4m)\frac{l}{2gd}$, the diameters and lengths being in feet.

$c = \sqrt{\frac{2g}{m}}$ in equation $v = c(ri)^{\frac{1}{2}}$.

J. T. Fanning, C.E.

Vol. XIX, p. 112.

PLATE XI—FANNING ON FLOW IN PIPES.

In computing for loss of head, or "friction-head," as it is termed, we have frequently to use the formulas when velocities are from 10 to 12 feet per second, and when computing for total capacities of pipes, the velocities are very great, and formulas with constant co-efficients give misleading results.

Mr. Brush has applied these formulas of Table No. 3 to a 20 inch pipe within the very limited experimental range of velocity from 2-feet to 3 feet per second, and yet it appears, as we scan over the table, that none of them were sufficiently well adapted to his case. He finds D'Arcy's formula, which his foot note says is made special for a 20-inch pipe, to best fit his experimental velocities and volumes. He has used it with the co-efficient a constant, and we see the results for volumes in Table No. 4, where, when velocity was 2 feet and measured volume at rate of 2 848 000 gallons per day, the formula gave 2 743 050 gallons, about 100 000 gallons short. When velocity was three feet and measured volume at rate of 4 255 200 gallons per day, the formula gave 4 297 580 gallons, about 42 000 gallons in excess.

If we plot the experimental and theoretical curves for this one limited series, we will find their lines to cross, and at the same time it appears the D'Arcy co-efficient is 11 per cent in excess of the co-efficients of the following formulas, and consequently gives, in this case, volumes about 11 per cent. nearer the true volumes than the others. If the co-efficient had been varied with variation of velocity as experience indicates proper, then the experimental and theoretical curves of velocities and volumes would have been respectively identical.

My diagram and the table of co-efficients taken therefrom has been very suggestive in a long practice, and has simplified many problems, not only relating to smooth pipes, but to conduits and sewers, that without its teachings would have proved very intricate. By similarly plotting for rough pipes and open channels, the new positions for their co-efficient curves are readily suggested.

A table of co-efficients for smooth pipes need not be very extensive for ordinary practice, and one page of the note book will give ample place for a series covering the range of pipes from 4 to 48 inches diameter.

The c of the Chezy formula is the symbol of a compound factor, but is readily deduced from the co-efficients of variables as diagramed. If we take $\frac{1}{m}$ as the symbol of the variables, such as are determined by experiment, then we find c to equal $\sqrt{\frac{2\ g}{m}}$.

If $c = 100 = \sqrt{\frac{2g}{m}}$ then $m = \frac{64.4}{100^2} = .00644$, and for other values of c we have $\frac{64.4}{m} = c^2$.

Thus the series of c may be diagramed or tabled as fully as the series of m for either pipes, sewers or open channels.

Toward the close of our morning session the discussion had turned to

The Use of Water Mains for Fire Purposes.

A matter of general interest in that line happened in one of the large cities in the Northwest during the last winter.

There were two large fires within one week in the wholesale district in St. Paul, at each of which an extensive warehouse with contents was destroyed. The division of the water mains controlling the fire service of this district extends northerly back from the river five blocks, and from east to west, parallel with the river, seven blocks.

The water is supplied by gravity, a 20-inch pipe coming to the northeast corner, and a 12-inch pipe to the northwest corner of the district. There is a 16-inch main along the east side, a 12-inch main along the north side, and a 12-inch main along the west side of the district, and this district is gridironed across in all its streets with 6 and 4-inch pipes, and has fire hydrants at each street corner.

At each of the fires nine steam fire engines were in use, and it was claimed that a number of the engines were occasionally unable to get a sufficient supply of water, and that by their suctions they were occasionally causing vacuums in the hydrants, and that the force and volume of the fire streams were thus diminished and ineffectual at times.

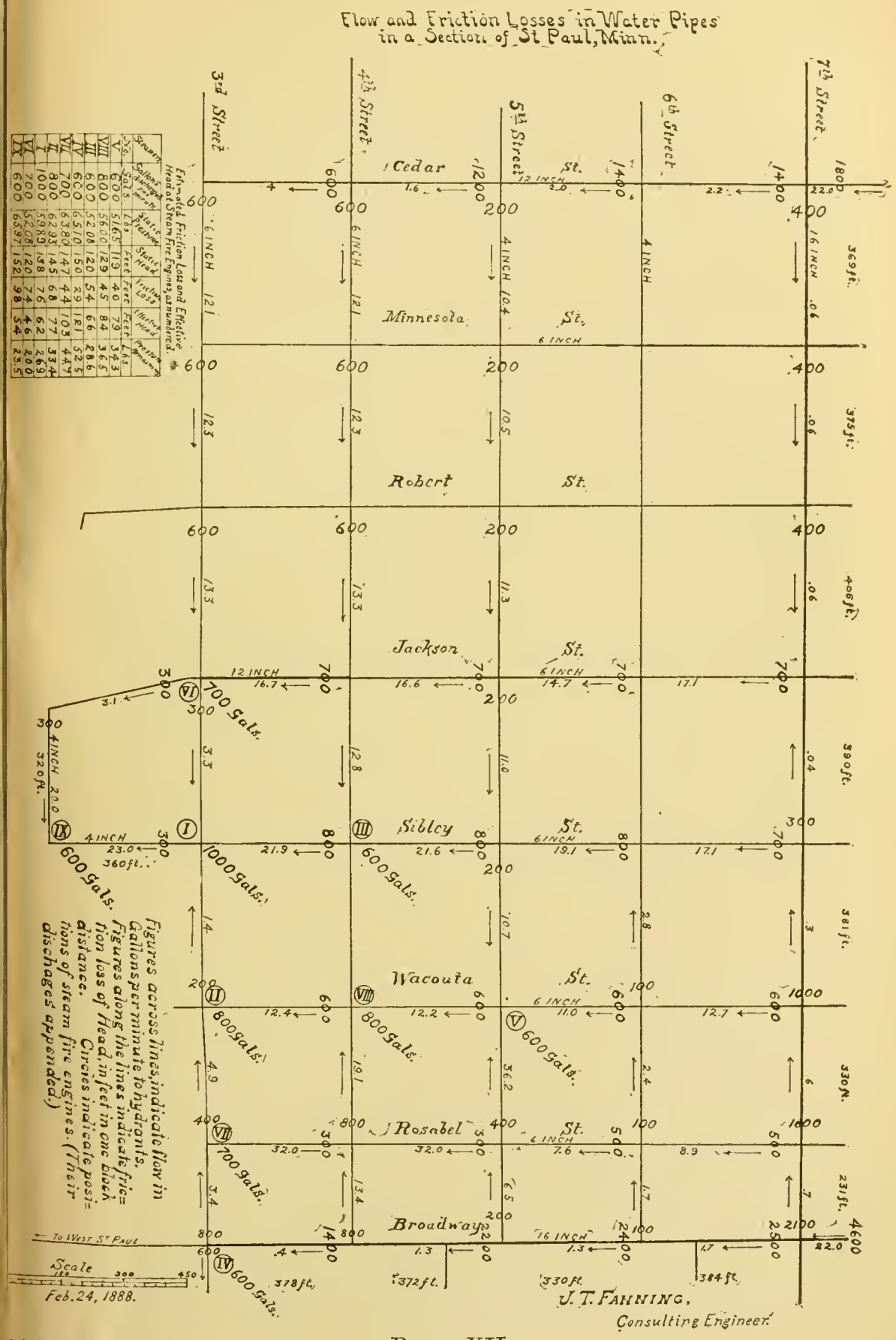
This claim led to the impression that the water mains in the district had not capacity to supply the fire steamers in use, and to severe criticisms of the Water Commissioners, and also to an immediate raising of fire insurance rates in the wholesale district.

The Water Commissioners believed they were unjustly blamed, and asked the speaker to investigate and report to them the capacity of the mains in the district.

Considering the climatic conditions of St. Paul's February, the immediate investigation could not readily be other than a theoretical examination of the conditions of flow in the mains, and there became involved here some of the interesting questions which Mr. Brush has suggested in his tables of experiments and theoretical flow in mains.

The static head of water under the condition of domestic consumption at that time was determined in the wholesale district by gauges, and found to be at the location of the lowest hydrant 152 feet head, and at the highest hydrant 119 feet head, with a mean head in the immediate vicinity of the ruins of the two burned warehouses of 135.5 feet, or 58.8 pounds pressure.

The capacities of the steam fire engines were claimed by those in charge of them to be as follows: four steamers, 600 gallons; two steam-



Stream	Gallons per minute	Static Pressure	Static Head	Friction Loss	Effective Head	Pressure remaining
	Gals.	Lbs.	Feet	Feet	Feet	Lbs.
V	600	516	119	40	79	34.3
VIII	800	560	129	45	84	36.5
III	600	520	120	54	66	28.6
IV	600	510	150	29	121	52.5
VII	700	380	147	44	103	44.7
II	800	693	145	68	77	33.4
I	1000	583	138	76	62	26.9
VI	700	508	120	74	46	20.0
IX	600	597	152	98	54	23.5

Estimated Friction Loss and Effective Head at Steam Fire Engines appended. *

Vol. XIX, p. 114.

PLATE XII.

ers, 700 gallons; two steamers, 800 gallons; and one steamer, 1 000 gallons; making a total for the nine steamers of 6 400 gallons per minute. As there were at the fires thirteen streams in play the greater part of the time, this would give to each stream an average of 492 gallons per minute. As I had found in tests of hydrant streams under high gravity pressure that 30 cubic feet, or 225 gallons per minute from a 1¼-inch nozzle made an excellent fire stream, I believed the capacities of the steamers to be stated much higher than their regular working capacities, but decided to base the computation on the claimed discharge of water as a possible excessive condition.

On a skeleton map of the pipes of this district was figured the estimated volumes of flow from the three large surrounding mains into and through the cross-distribution mains toward the hydrants in use, displaying at a glance the assumed volume of flow along each block and in each pipe. (See Plate XII).

The loss of pressure along each block in the direction of the hydrants was then computed and summed to obtain the total loss at each hydrant in use by a steamer, the estimated volumes of domestic use being included in computing frictional losses.

The computation indicated that even though the steamers reached the improbable discharge of 6 400 gallons per minute, there remained throughout the district a mean pressure of at least 34 pounds per square inch at the hydrant nozzles, and therefore, if not impossible, it was certainly improbable that there was any approach to a vacuum in any hydrant in use, if its valve was fully open. The claims of the Water Commissioners that the failure of the fire streams was not chargeable to any defect in the water mains being still opposed, the Commissioners asked that a thorough test be made of the mains with the fire steamers again in position as at the time of the last large fire, and that gauges be applied on the mains and hydrants, and their pressures be observed.

After much public discussion of the question, such a test was decided upon, and Mr. J. W. Hill, M. Am. Soc. C. E., was selected to direct the same.

Such a thorough test was made a few weeks since with nine steam fire engines and fifteen streams in use, and we are now waiting for the report, which we are confident will give voluminous experimental data that will be valuable to members of this society and to the profession.

The active test was continued during a period of four hours, which gave ample opportunity to observe the readings of the gauges and the actions of the steamers and their streams. The engines were worked so vigorously during the first hour that the highest readings of their respective water gauges were noted to range from 200 to 275 pounds, and yet it was evident by the readings of the pressure gauges on the hydrants that the combined discharges of all the engines was not so great as their claimed rating, which I have previously stated.

Occasionally, with 40 to 45 pounds pressure on the hydrant gauge, as a steamer was speeded up, it would apparently be short of water, and its stream would diminish.

Observing on such occasions that the pressure rose in the hydrant gauge and diminished in the steamer's water gauge proportionally as the hose stream diminished, it was evident that the supposed vacuum, that had caused so much discussion, was not in the hydrant, but was probably between the suction valves and the pump piston, and hence a conclusion that the piston speed at its maximum was greater than water could follow through the grid of the valve seat, and that the pump chamber was but partially filled with water at each stroke of the piston.

To further test whether the hydrant gauges were deceptive as to pressure, I directed that a hose be attached to the spare nozzle of a hydrant while the steamers were in full work.

A direct pressure stream was thus obtained, which accounted for the pressure indicated by the gauge, and this direct pressure stream through a short hose was about equal in force to the stream from the adjoining steamer delivered through a longer hose.

I will not attempt to offer general deductions from memory of my notes or observations. Mr. Hill has the complete records of observations of the forty-eight gauges for the preparation of his report, and I anticipate that the report will be so full and able that it will put us in possession of many new and valuable facts relating to the frictions of flow of water through pipes in fire service.

A. Fteley, M. Am. Soc. C. E.—After hearing the paper just read, it occurs to me that aside from its merit in presenting numerous data collected from a long series of observations, it has the additional advantage of giving us all the results obtained, as they are collected, regardless of the fact that they may not have been, at times, as satisfactory as might have been expected.

This course is very commendable, and in the present case, the usefulness of the results furnished is enhanced by the well-known care and ability displayed by the author in the conduct of his work.

Not having been connected lately with the management of any water distribution, I regret not to be able, at present, to give any new fact of interest in connection with this question, but I will ask Mr. Brush to give us, if possible, some additional figures in regard to his observations of the flow of water through a 20-inch pipe.

The results furnished by him give the average of a great many observations. Some of the individual results may vary considerably from the average, either one way or the other, and the knowledge of the range and distribution of these variations would be interesting.

A short description of the particular conditions obtaining at the time that such variations were observed, would also be of great interest.

In the conduct of experiments of this character, the more accurately the conditions prevailing at the time of the observations are described, the more convincing are the deductions made and the more confidently they can be received and applied.

The remarks just made by Mr. Fanning in regard to the difficulty of harmonizing the results furnished by various experimenters for the flow through pipes of the same diameter, illustrate well the probable fact that the difference of the results obtained in apparently identical circumstances, is due to the existence of individual conditions, which have been ignored or incompletely described by the experimenters.

I consequently believe that the author of the paper would add to its interest by giving a tabulated list of his results, with such remarks as might be of interest in connection with them. The reader would then be able, by a study of individual observations, to select those which are more applicable to his case, and to draw his own conclusions.

Before closing these remarks, I may be permitted to recall here and to deplore the death of the modest and able engineer, to whom our profession is indebted for the formulation of his admirable research on the flow of water, W. R. Kutter, of Switzerland.

Mr. Brush.—The information requested by Mr. Fteley I will be very glad to furnish. Our system is to take the record every hour during the twenty-four hours of the day. Therefore we know the velocities at which we were pumping at different times during different days. From the experience of five years and taking the average of the results obtained when running at given velocities during this whole period, I have obtained the data presented in my paper. I will present hereafter, as requested, a more definite statement of the number of these experiments.

Mansfield Merriman, M. Am. Soc. C. E.—Were the velocities which are given in the first table of the paper determined by displacement of the pump?

Mr. Brush.—By the displacement of the pump. This was compared during the first two months of our supply with the amount of water delivered into the reservoir. We knew exactly the size of the reservoir and its capacity for every inch of height. We found that the difference was not more than 3 per cent. I have allowed 5 per cent., extending over the whole period of five years, as loss by pump displacement, *et als.*

Oberlin Smith, M. Am. Soc. C. E.—I would like to ask Mr. Brush a question or two.

First.—Whether he has made experiments with the ordinary cast-iron main joints, following out the average loss? It seems to me that the

figures in the last column of the first table on page 103 are remarkable. What I want to know is, about what proportion of this 64 per cent. is due to the leakage of the cast-iron main; whether he has an idea of the average loss due to the leakage of the cast-iron main, which of course is a certain part of this 64 per cent.?

The other question is whether he has experimented with large wrought-iron pipes in place of cast-iron, and, if so, if he finds much difference?

Mr. BRUSH.—The 64 per cent., as I explained this morning, was the loss during the first quarter, and that was due to the amount of water that was blown off to clean the main. As I said, these percentages are only relative. There is a constant amount of water which has to be wasted from every distribution system in order to keep the pipes clean for fire purposes, for sprinkling streets, etc. Therefore, the earlier percentages in the table are of little value. As a matter of fact, the preceding column in the table gives you a better guide as to the actual amount of loss than the percentages in the last column.

I have had considerable experience with small wrought-iron pipe buried in the ground, and it has been unsatisfactory. There is great lack of uniformity in its durability. On the same line of pipe one portion will last a long time, while another portion will soon wear out. The difference seems to depend on the soil in which the pipe is laid. Ashes especially will soon destroy wrought-iron pipe, often during the first year. My observation on large wrought-iron pipe, underground, leads me to distrust it for water mains in this section of our country. There is a marked difference in the climatic conditions of the Pacific Coast, where this pipe has been successfully used, and our Atlantic Coast.

G. W. PEARSONS, M. Am. Soc. C. E.—I would say that my experience in the use of perhaps 200 miles of cast-iron mains leads me to think that no important leaks can exist without showing at the surface, unless some access is made by the leak to subterranean drains, either natural or artificial.

In one case I dug up a place where, in dry loamy ground, the water had come to the surface and formed quite a pool; the leak, when found, proved to be a single stream about as large as a fine cambric needle, certainly less than one-fortieth of an inch in diameter.

In another case a line of 20-inch pipe, with Robbins' patent joints, showed for over a mile by the settlement of the newly-filled trench the location of almost every joint; these soon closed, and the pipe became so tight as to allow the ground to dry with the rest, and in subsequent digging for connections no leaks or wet places were discovered. This was in dense clay soil, where a very slight leak would have the effect of keeping the ground saturated.

In another case a 9-inch pipe carrying water from a mountain lake, so clear that the proprietors of the works scouted the idea of any sediment having ever entered the pipe, became entirely closed by the accumulation of soft silt in the pipe.

I think that it is safe to estimate that not the hundredth part of one per cent. of discovered loss is due to actual leakage through the pipe or its joints; but a reverse action taking place in all valves and connections, these almost certainly become leaky by use. I have seen thirteen valves of from 4 to 6 inches closed for repair, and a steam fire engine have all it could do to keep the trench clear, and this is not an isolated or exceptional case. Service pipes have access to drainage, and their leakage is only reported where it is annoying; the still joints of the main lines, if leaky at first, soon close by silt.

Mr. BENZENBERG.—I suppose I could add a word as to what we found here in seeking for the causes of the enormous wasteage of water in Milwaukee. The regulations here require that wherever a street is to be paved the water service pipes and house drains must be laid to each lot.

In looking for leaks and waste of water with the aid of the Bell waterphone, it was discovered that in one square or block, out of four service pipes which had with the drain pipe been laid to the curb line, and which had never been extended, there being no improvements of any kind on said lots, three were discharging water without any indication on the surface.

On digging down it was found that the stop-cocks at the curb had been left wide open, and the water was discharging direct into the drains, which had been laid at the same time, evidently from the very first minute that the pipe had been laid, the plumber having left the stop cock open when he had finished the work.

As I said, there were three such cases in one block. This of course led us to look for wastes in a direction new to us.

SUMMARY BY MR. BRUSH, SEPTEMBER 13TH, 1888.

Since this paper was read and discussed at our Annual Convention last June, I have obtained from Warren E. Hill, M. Am. Soc. C. E., the following valuable data:

EXPERIMENTS ON THE DELIVERY OF WATER FROM PIPES, MADE BY THOMAS F. ROWLAND, CONTINENTAL WORKS, GREENPOINT, BROOKLYN, N. Y., JULY 19TH, 1883.

These experiments were made at the works of the New York Gaslight Company, Twenty-first street and Avenue A, on ¼-inch, ½-inch and 1-inch wrought-iron steam-pipes of varying lengths, and under varying pressures. The object was to ascertain whether Eytelwein's formula for the flow of water in pipes, is applicable to pipes of small diameter, and if it is not, to make the proper correction in the constant.

Eytelwein's formula is as follows: $Q=4.72\sqrt{\frac{D^5 H}{L}}$ in which Q is the quantity of water delivered per minute in cubic feet; D, the diameter of the pipe in inches; H, the head of water in feet, and L, the length of pipe in feet.

The water was delivered from an accumulator by a 2½-inch pipe, and the pressure was taken from an Ashcroft gauge placed near the junction of this pipe, and the pipes to be experimented upon. A 2½-inch globe valve, placed between the gauge and the accumulator, served to regulate the pressure to a nicety. The water was delivered into a barrel under full head, promptly and without any loss. It was weighed on a Fairbank's scale; the time was taken with a "stop watch."

These experiments show that the constant (4.72) in Eytelwein's formula should be as follows:

For	¼-inch pipe, 30— 60 feet long	6.90
	½-inch pipe, 30— 60 "	6.98
	1-inch pipe, 30—100 "	4.85

This is for straight pipe. It was found that two elbows or a return bend would reduce the delivery 2 or 4 per cent.

TABLE.

Experiments.	Size of Pipe in inches.	Length of Pipe in feet.	Pressure in pounds per square inch.	Head of Water in feet.	Time of observation in seconds.	Water delivered during observation, pounds.	Water delivered during one minute, in pounds.	Water delivered per minute in gallons.	Water delivered per minute in cubic feet.	Delivery as per Formula.	Radical of the Formula $\sqrt{\frac{D^5 H}{L}}$	Corrected Constant.
1	¼	30	68	188	60	32	32	3.8	.512	.369	.0782	6.54
2	¼	30	100	277	60	39	39	4.7	.624	.447	.0949	6.57
3	¼	30	119	330	60	44	44	5.3	.708	.486	.1030	6.83
4	¼	60	72	200	60	25	25	8 0	.400	.269	.0570	7.01
5	¼	60	95	264	60	29	29	3.5	.464	.302	.0650	7.25
6	¼	60	119	330	60	33	33	3.9	.528	.344	.0730	7.23
7	½	32	65	180	90	264	176	21.2	2.81	1.97	.419	6.70
8	½	32	90	250	75	268	214	25.8	3.43	2.34	.496	6.91
9	½	32	120	333	60	259	259	31.2	4.01	2.79	.570	7.00
10	½	64	70	194	60	135	135	16.2	2.16	1.45	.307	7.03
11	½	64	100	277	60	163	163	19.6	2.61	1.73	.367	7.11
12	½	64	120	333	60	180	180	21.7	2.88	1.90	.403	7.14
13	1	31	70	194	24	295	737	88.8	11.80	11.80	2.5	4.72
14	1	31	100	277	15	222	888	107.0	14.20	14.11	2.99	4.75
15	1	31	120	333	15	246	984	118.5	15.74	15.43	3.27	4.81
16	1	63.5	70	194	30	273	546	65.8	8.73	8.26	1.75	5.04
17	1	63.5	100	277	20	220	660	79.5	10.50	9.86	2.09	5.02
18	1	63.5	120	333	15	187	748	90.0	11.97	10.81	2.29	5.22
19	1	97	70	194	45	305	407	49.0	6.51	6.65	1.41	4.61
20	1	97	100	277	30	250	500	60.2	8.	7.97	1.69	4.74
21	1	97	120	333	30	278	556	60.7	8.89	8.73	1.85	4.79

I have also obtained from G. W. Pearsons, M. Am. Soc. C. E., the blue print of a diagram which appears in the Transactions of this Society for February, 1876, and which diagram, with some modifications, was subsequently published in the *Engineering News* for December 24th, 1887. In a letter accompanying said diagram, Mr. Pearsons makes the following statement:

"I have noted on the blue print you ask for, your quantities, and would add the following for your consideration. We found that our other plunger pumps here required packing almost when new; that a very thin film of water around an unpacked plunger made a serious loss, and my experience with Worthington pumps leads me to think that you will find, on trial (pump under pressure against closed valves), that your loss is more than 5 per cent.; add to this uncertainty the 2 per cent. allowed in my diagram, and you may find that they do not vary seriously; another thing in this connection, I think, is that your pipes are smoother and evener than those we usually get here; my aim has been to reach as far as I could in quantities and be safe, but to be safe, as I have never known a case where a less friction than estimated would be injurious. As to the Quindardo mains, so far as I was able to observe last year, the flow corresponded very closely with the diagram. I hold that very little leakage occurs in the pipe itself; but in making such a test as you did, indicating a 70 000 gallon loss by leakage, I should want to be very sure about the stop valves. I have seldom found a valve tight after much use; have seen thirteen gates from 4 to 6 inches closed, and a large steam fire-engine have all it could do to keep the leakage clear, and have seen dozens of similar instances more or less closely approximating to this."

From the interesting report of Dexter Brackett, Assistant Engineer of the Boston Water Works, dated May 1st, 1882 (see City Document 124), I quote the following in relation to the Charlestown Works, built in 1864–65:

"The main pipes of this district are made of wrought-iron, coated inside and out with cement mortar. Fifteen years' use proved them defective, and they have been partially replaced by cast-iron pipes. It was expected that many leaks would be found from the cement-coated pipes, but these expectations were not realized. In a few cases the closing of a main gate, after all the services had been closed, showed a waste of from one to three hundred gallons per hour from the main or from the services between the main and the sidewalk shut-offs, but no leaks were discovered of sufficient magnitude to warrant the expense of uncovering a long line of main pipe."

From his report we also learn that in five different districts in which the population varied from 2 000 to 3 000 people, and where the ordinary

night draft had been running at the rate of from 50 to 60 gallons per head per day, Mr. Brackett was enabled, by carefully restricting waste, to reduce this night draft to the rate of from 9 to 15 gallons per head per day.

Replying to the inquiry of Mr. Fteley, I would say that the table of the results in pumping water through our 20-inch main, which appears above in this paper, was obtained from a consideration of six hundred and seventy-three observations. There are eight vertical columns in that table showing these results.

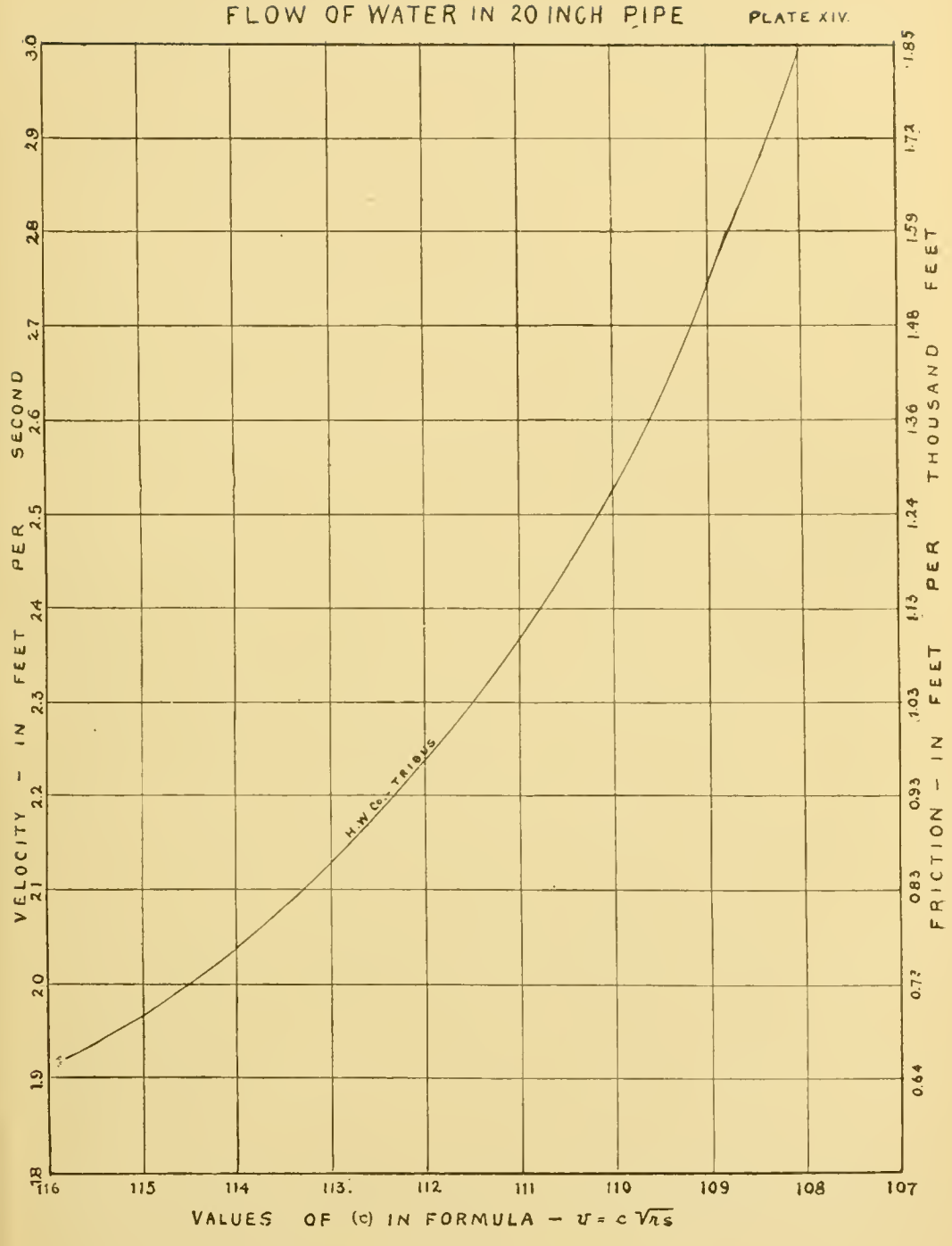
The following is a statement of the number of observations taken in each case, and the maximum and minimum results.

Column.	Given discharge per day in gallons.	No. of observations.	Range of observations from discharge as given. Below.	Above.
1	2 848 000	75	6 per cent.	3 per cent.
2	3 165 200	135	5 "	2 "
3	3 354 250	125	4 "	2 "
4	3 566 800	125	4 "	2 "
5	3 804 400	110	2 "	2 "
6	3 904 200	75	2 "	4 "
7	4 116 750	25	6 "	1 "
8	4 255 200	3	1 "	1 "

Mr. Fanning, in his comprehensive discussion, has furnished a diagram of co-efficients of flow in pipes under pressure for different size pipes and different velocities. It will be interesting to compare his theoretical results with those actually obtained by the Hackensack Water Company.

The following table shows this comparison, and also shows the results obtained from Mr. Pearson's diagram of flow of water in long pipes above referred to, as well as results obtained from the experience of Mr. Fteley:

Column.	Velocity in feet per second.	Friction Head per 1000 Feet. H. W. Co. R	Fanning.	Pearson.	Fteley.
1	2.00	0.729	0.791	0.928	0.773
2	2.24	0.876	0.983	1.136	0.970
3	2.36	1.029	1.085	1.288	1.076
4	2.52	1.316	1.232	1.401	1.226
5	2.68	1.337	1.388	1.557	1.387
6	2.76	1.490	1.464	1.651	1.470
7	2.92	1.645	1.629	1.852	1.647
8	3.00	1.797	1.713	2.017	1.738


FLOW OF WATER IN 20 INCH PIPE
PLATE XIV.
VELOCITY - IN FEET PER SECOND
FRICTION - IN FEET PER THOUSAND FEET
H.W. Co.'s TABLES
VALUES OF (c) IN FORMULA - $v = c\sqrt{rs}$
3.0
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2.0
1.9
1.8
1.85
1.73
1.59
1.48
1.36
1.24
1.13
1.03
0.93
0.83
0.72
0.64
116
115
114
113
112
111
110
109
108
107

Effective Head in feet	43	55	66	77	89	100	112	123	135
Discharge United States Gallons in 24 hours	2 726 600	2 848 000	3 165 200	3 354 250	3 566 800	3 804 400	3 904 200	4 116 750	4 255 200
Velocity in feet per second	1.92	2.00	2.24	2.36	2.52	2.68	2.76	2.92	3.00
Friction in feet per 1 000	0.614	0.770	0.932	1.072	1.245	1.387	1.563	1.719	1.865
15 000 FEET FROM ENGINES. 100 000 UNITED STATES GALLONS REMOVED.									
Discharge	2 626 600	2 748 000	3 065 200	3 254 250	3 466 800	3 704 400	3 804 200	4 016 750	4 155 200
Velocity	1.858	1.943	2.168	2.302	2.452	2.620	2.691	2.841	2.939
Friction	0.567	0.728	0.8714	1.02	1.177	1.3.5	1.483	1.626	1.79
70 000 FEET FROM ENGINES. 100 000 UNITED STATES GALLONS REMOVED.									
Discharge	2 526 600	2 648 000	2 965 200	3 154 250	3 366 800	3 604 400	3 704 200	3 916 750	4 055 200
Velocity	1.828	1.873	2.097	2.231	2.381	2.549	2.620	2.770	2.868
Friction	0.548	0.674	0.814	0.958	1.11	1.256	1.405	1.546	1.706

In reply to a question raised during the discussion at the Annual Convention, I stated that the value of the tables on page 103 largely consisted in the fact that we were driven to the necessity of greater vigilance and more careful examinations to determine waste. The truth of this remark has been verified. Since that meeting careful and repeated night inspections have developed several leaks in our mains in the high service district, the aggregate loss from which leaks amounted to about 125 000 gallons per day.

During the discussion there seems to have been an inference drawn from my paper to the effect that I thought part of the leakage was through the pipe. I did not intend to convey that idea at all, but rather that in every extended distribution system there were more or less cases of defective joints due to carelessness in the original laying of the mains, or to water-hammer after the mains are laid, and that the waste from such joints or from cracked pipe frequently did not appear on the surface of the ground.

There is one point, presented in this paper, that has not been considered in the discussion, and that is the maximum amount of water that can be economically pumped through a given main and the relative cost at different velocities. It seems to me that the results of actual experience in this matter would be information of very great practical importance.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS.

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(Vol. XIX.—October, 1888.)

THE THEORY OF AQUA AMMONIA ENGINES, WITH RESULTS OF TESTS UPON AN ENGINE OF THIS TYPE OPERATING AN EDISON INCANDESCENT LIGHTING PLANT.

By EDWARD E. MAGOVERN, Jun. Am. Soc. C. E.

READ SEPTEMBER 5TH, 1888.

The writer, in the course of professional work, has recently been called upon to investigate a system of power development, in which the vapor of aqua ammonia has been substituted for steam with remarkable findings, both as to economic and potential efficiency, and desires to communicate these results, together with an examination as to how far our present knowledge of the science of thermodynamics will explain or attempt to contradict the facts experimentally obtained.

The fact that the steam engine, as a prime mover, realizes in useful effect but about one-tenth of the available energy stored in the coal as heat, is so well known, that its mere statement, even to a layman, is deemed almost unnecessary. To the professional Engineer, however, whose function may be defined as the creating of profit for capital, the realization that by reason of the intrinsic physical properties of the medium commonly employed to convert the heat of the coal into work,

nine-tenths of the available energy of the fuel is wasted, awakens in the mind the question: May there not be, among the many liquids, and among the many gases whose properties have been determined and whose law of changes fixed by the patient labors of our physicists and chemists, some medium, liquid or gaseous, or both, which can be employed to transmit the heat from the coal to the engine, there to be converted into useful energy, without this enormous loss?

The question, father to the action, leads to a consideration of the causes of loss; and the possible remedy, the employment of a liquid having a lower latent heat of vaporization, instantly suggests itself. These, fortunately or unfortunately, are many in number, the principal of which are given in the subjoined list:

	Latent heat of vaporization in British thermal units.
Anhydrous ammonia	895
Alcohol	364 to 372
Bisulphide of carbon	156 to 212
Carbonic acid	298
Sulphuric ether	163 to 174
Turpentine	137 to 184
Mercury	157
Chymogene	175

Doubtless, it would appear to a novice, with the exception of anhydrous ammonia, whose latent heat is less than ten per cent. lower than that of water, a promising list. To the experienced Engineer, however, the inflammability, explosive or poisonous qualities of the vapor, the difficulty of confinement, the comparatively light density of the evolved vapor requiring a comparatively larger volume to carry the equivalent available heat, thus necessitating the employment of larger cylinders and other detractive qualities, make the outlook less promising, and the anticipations of enormous theoretical saving, sink, overwhelmed by difficulties, which, correctly or incorrectly, are considered insurmountable. Nor does the attempt at discouragement of the inventor cease here. Theoretical deductions from the infant science of thermodynamics not yet fully understood, and not yet subjected to crucial test, are hurled in phalanx against any attempt to shake the wasteful tyrant, steam, on his throne.

In a recent article, relevant to the subject in hand,* that eminent authority, Professor R. H. Thurston, M. Am. Soc. C. E., from whom

*Steam and its Rivals, Forum, May, 1888.

in former years the writer had the pleasure and profit of receiving instruction, sums up as follows:

"Comparing alcohol, ether, bisulphide of carbon and chloroform, the most promising of the available fluids, with water, and its vapor, steam, it was found that, as the well-known laws of thermodynamics indicate, all were of precisely equal efficiency if worked in perfect engines, within the same range of temperature, with no waste occurring other than that which has already been described in the earlier part of this paper as unavoidable."

To these conclusions, and to that portion of the article itself which treats of the efficiency of heat engines, and as far as the analysis is carried, the writer cannot except. But to conclusions based primarily on a consideration of the performance of vapors in the cylinder only, and then applied to the complete cycle of operations incident to an engine, with the fluid supplied and evaporated in a separate vessel, the reception of heat, being, not as provided for by Rankine, wholly at one absolute temperature, T_1, and its rejection wholly at another absolute temperature, T_2, but in a more complex and less readily understood manner, to these conclusions the writer objects, such application being incompetent, irrelevant and immaterial.

The fact that the efficiency of all vapors, in the cylinder, received at an absolute temperature T_1, and rejected at a lower absolute temperature T_2, can be approximately* expressed as Rankine has done (cylinder efficiency only, let it be remembered) by $\frac{T_1 - T_2}{T_1}$, is not denied.

The conclusions above quoted it is understood were arrived at by Messrs. Gantt and Maury,† after a thermodynamic study of the fluids spoken of; expressed in other words, the conclusions mean, that at the same pressure and temperature there is the same amount of heat in like volumes of different gases. This correct conclusion might have been arrived at by a mere study of the physical properties of the gases without reference to thermodynamics, for Delarive and Marcet (Annales de Physique et de Chimie, tome 35, p. 5, tome 41, p. 78, tome 75, p. 113) and Heyward (Edinburgh Phil. Journal, Vol. X., p. 351) have shown

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* My friend, and former instructor, De Volson Wood, Professor of Mechanical Engineering at Stevens Institute, was, I believe, the first to demonstrate that this formula is only approximately true.

† Van Nostrand's Magazine, 1884.

"that in saturated gaseous media the volume is directly as the latent heat;" again, Avogardo's law states: "in like volumes of all gases there is the same number of molecules," and as the product of the molecular weight and specific heat of all gases give constant result (proved by Du Long and Pettit, and accepted by such authority as Clerk Maxwell in his "Theory of Heat") it follows, as stated by Clerk Maxwell and by Cooke, in his "Chemical Physics," that there is the same amount of heat in like volumes of different gases.

One vapor, therefore, having had the necessary heat imparted to it and entering the cylinder, will, practical considerations aside, be as good for power purposes as any other vapor; and hence, Rankine's formula $\frac{T_1 - T_2}{T_1}$, containing no variable of gaseous composition, the only variables being the limits of temperature, is undoubtedly, for cylinder, or as it may be termed, gaseous, efficiency correct. Rankine, too, appreciated the above, and* states that "to fulfill strictly the above conditions of maximum efficiency between given limits of temperature, the elevation of the temperature of the fluid must be wholly performed by compression and the depression of its temperature wholly by expansion, operations which are in many cases impracticable, from the great bulk of cylinders which their performance would require." The most radical of the adherents of steam will hardly believe that a formula applicable, as Rankine has shown, to the elementary heat engine, should be applied to the class of engines mentioned by Professor Thurston. The formula, as above given, may be taken to represent only the expenditure of heat; its reception in most engines being apart from its point of expenditure, must therefore be separately considered, and if it can be shown that the reception of heat is, like its expenditure, independent of the fluid employed, the question of employing any vapor to replace steam as a medium for transmitting the heat of the coal to the engine, to be converted into power, is settled forever in favor of steam by reason of the absence of detractive qualities spoken of above.

A moment's thought will be sufficient to convince one, familiar with the physical properties of liquids and gases, that the reception of heat does involve a consideration of the nature of the fluid, otherwise the specific heat of all substances would be unity.

Again, practical considerations of the furnace and boiler efficiency

* Steam Engine and Prime Movers (1876 Ed.), p. 344, art. 268.

enter into a correct consideration of the resultant efficiency of the system, and it will readily be granted that the bare formula $\frac{T_1 - T_2}{T_1}$ involves no such considerations. To further exemplify this point, let two sister engines, each designed of maximum efficiency, be taken, and let it be granted that such a design is possible. Into one boiler we place liquid anhydrous ammonia, into the other water. Exposed to the ordinary temperature of the air in this latitude sufficient heat will be furnished by the atmosphere to generate gas within the ammonia boiler, and let it be supposed that the pressure so induced has, by regulating the volume of liquid ammonia introduced, become equal to one atmosphere; the water boiler, however, at any temperature below 212 degrees, will not give off sufficient vapor to create any pressure, and thus both boilers are at atmospheric pressure. Let the condensers of both these engines be located in the polar zone at a time when the temperature there is as low as — 37 degrees Fahr., and assume the condensers to be connected to the engines by an amply large pipe. Open the throttle valve of each engine, start the fly wheel, and if our reception of heat is the same in both cases, the result should be the same. But it is not; the ammonia gas discharged and entering the condenser is reduced to — 37 degrees in temperature and liquefies, relieves the exhaust side of the piston by condensation, and the motion will continue as long as the temperature of the condenser is maintained below — 37 degrees, and sufficient fluid injected into the boiler to permit the generation of an adequate supply of gas to fulfill the above conditions. The other engine has made a turn or two, and stopped; the temperature of the air is not sufficient to generate vapor for the operation of the engine. Have we attained perpetual motion without expenditure in the ammonia engine? Not at all; we have merely utilized the heat in the atmosphere, derived mainly from the sun, as Ericsson in his solar engine utilizes the sun's heat, though in another way.

Suppose we kindle a fire under the water boiler and in sufficient quantity to make its engine's motion and power equal to that of the ammonia engine (the exhaust vapor being condensed and frozen at the condenser), we then have

$$\frac{T_1 - T_2}{T_1} \text{ for steam} = \frac{T_1 - T_2}{T_1} \text{ for ammonia.}$$

Right; our engine or gas efficiency is the same. Is it obtained at

the same cost? No; by the amount of coal required to keep the steam engine going. The writer therefore concludes:

First.—That a thermo-dynamic analysis of the efficiency of fluid in any heat engine, save of the elementary type, should involve not only the efficiency in development of power at the engine, but also its efficiency in the reception of heat at the boiler.

Second.—That there is nothing in the science of thermo-dynamics, as understood to-day, which precludes the possibility of obtaining a less wasteful medium than water for the conversion of heat into mechanical energy.

The particular fluid which has of late years attracted the attention of scientific men as a substitute for water is aqua ammonia. This fluid, which is a mechanical solution of ammonia gas in water, possesses the following remarkable physical, chemical and mechanical properties:

1st. That while extremes of cold or pressure, or both, are necessary to liquefy ammonia gas, forming anhydrous ammonia, ammonia gas is readily absorbed by water and held, as one has seen very fine particles of solid matter held in water, mechanically. Therefore, the least application of heat causes the discharge of the gas, which has no latent heat of gasefication, not having become liquid. If an ordinary bottle of medicinal aqua ammonia be held at a temperature of 98 degrees for two or three hours, sufficient gas will be generated to force the cork from the bottle.

2d. The density of ammonia gas and steam are nearly equal, and therefore, at the same temperature, ammonia gas contains nearly the same amount of sensible heat, the only heat which can be utilized in the engine, per pound, as does steam.

3d. The vapor is non-explosive, and can be confined as readily as steam.

An analysis of the efficiency of the fluids, steam and ammonia, may be undertaken as follows:

Let T_1 and T_2 = the temperature of evaporation of aqua ammonia and steam respectively at the same gauge pressure.

= temperature of feed, for comparative work taken the same for the aqua ammonia as for steam.

W = weight of aqua ammonia required to produce $\left(=\frac{W_2}{q}\right)$

W_2 = weight of ammonia gas required per hour, about ten per cent. more than

W_1 = the weight of steam required per hour.

q = per cent. of ammonia present in W.

q_1 = " " " in feed liquid.

q_3 = " " " present in working vapor, reckoned $(W - W_2)$.

L_1 = latent heat of evaporation at gauge pressure for water.

L_2 = latent heat of evaporation at gauge pressure for the water of the aqua ammonia.

s_1 = specific heat of ammonia, taken as constant.

s_2 = " " steam " "

For Steam.

$(T_2 - t)\ W_1$ = *B. T. U.*, required with steam to raise W_1 pounds of water from temperature of feed to that of evaporation, and

$(T_2 - t)\ W_1 + W_1 L_1$ = total heat of evaporation of W_1 pounds of steam per hour.

For Ammonia.

$(T_1 - t)(W_1 - W_2) + W_2 s_1$ = *B. T. U.*, required at start with aqua ammonia to raise the liquid to the temperature corresponding to its gauge pressure.

$q_3 (W - W_3) L_2$ = *B. T. U.*, required per hour to evaporate the water at a temperature corresponding to gauge pressure,

then for a run of n hours we have for total heat expended

(1) $[(T_1 - t)(W_1 - W_2) + W_2 s_1] + [q_3 (W - W_3)] L_2 n + n W_2 s_1$.

It will be observed that the first quantity is independent of the time of run, and therefore in a comparative test, the longer the time run the greater should be the economy shown; this theoretical deduction is confirmed by an inspection of the results of the tests.

Again, practical experience with the fluid has shown that its loss of heat is very small, so that the pressure on the boiler is little reduced after standing under banked fires during the night.

The quantity $q_3 (W - W_3)$ representing the amount of water from the aqua ammonia evaporated has considerable influence in reducing the

theoretical efficiency of the fluid; the cause of this will be explained further on. The value of the quantity, $q_3\ (W - W_3)$ may be arrived at as follows: A sufficient quantity of the gas leaving the boiler is drawn off by means of an aspirator, into a coil surrounded by water, and sufficient pressure by means of an air pump put upon the coil to ensure absorption of the gas, after the liquid has cooled; the latter is now drawn off by means of a cock into a suitable vessel, and its specific gravity determined by means of a Beaumé hydrometer.

The following table, published for the first time through courtesy of Mr. George Shepard Page, President of the Washington Ammonia and Chemical Company, gives the percentage of ammonia present in mechanical solution corresponding to different readings of the Beaumé hydrometer at 60 degrees Fahr.

Reading.	Specific gravity.	Per cent. NH_3
26 degrees	.89744	28.8
24 "	.90909	24.9
22 "	.92105	21.1
20 "	.93333	17.3
18 "	.94595	13.6
16 "	.95890	10.0

The above analysis does not include, as will be apparent later, the heat lost by the spray liquid from the boiler. If it is desired to carry the analysis to this point, the following formula will be found applicable.

To the above notation add the following:

t_2 = temperature of spray at exhaust.
t_3 = temperature of exhaust.
W_3 = weight of spray per hour containing
q_4 = per cent. of ammonia present in the spray.
t_4 = resultant temperature of the mixture of spray and exhaust.

$$t_4 = \frac{[(W_2 - q_3\ W_2)\ s_1 + q_3\ W_2\ s_2)]\ t_3 + [(W_3 - q_4\ W_3) + s_1\ q_4\ W_3)]\ t_2}{[(W_2 - q_3\ W_2)\ s_1 + q_3\ W_2\ s_2)] + [(W_3 - q_4\ W_3) + (s_1\ q_4\ W_3)]}$$

we have, therefore, W_3 pounds of spray liquid reduced from temperature T_1 to temperature t_4 by the spray, which loss hourly will be, in *B. T. U.*

$$[(W_3 - q_4\ W_3)\ (T_1 - t_4)] + q_4\ W_3\ s_1\ (T_1 - t_4),$$

the total expenditure of heat for n hours equation (1) must, therefore, be increased by this quantity, giving

$$(1a.)\ [(T_1 - t)\ (W_1 - W_2) + W_2\ s_1] + [q_3\ (W - W_3)]\ n + n\ W_2\ s_1 + n\left[[(W_3 - q_4\ W_3)\ (T_1 - t_4)] + q_4\ W_3\ s_1\ (T_1 - t_4)\right]$$

Following a line of argument, similar, doubtless to the above, many

engines employing aqua ammonia have been devised, and from time to time attracted much scientific attention; the one difficulty in the successful adaptation of the vapor seems to have been to affect the absorption of the vapor without back pressure on the engine.

Dr. F. A. P. Barnard, late President of Columbia College, commenting upon the ammonia engine of Froment at the Paris Exposition, calls attention to the economy possible in the employment of ammonia vapor over that of steam, and regrets that Froment's engine, opening, as it were, a field of such promise, should fail owing to the back pressure on the engine induced by the want of complete absorption of the exhaust vapor.

Experiments have been continued from time to time, new engines planned, and still this one defect proved to be unremedied; so impressed was the writer with the necessity of providing an adequate method of absorption, that when attention was first directed to the engine to be described below, the first question asked was: "How is the absorption effected, and what back pressure is found on the engine?" The answer to this will be found in the results obtained as given below. Suffice it to say, that the introduction of the spray vapor is made at such a point that the heat of absorption is removed by practically surface condensation, and that the final rising of the condensed vapor through a volume of supernatant liquid liquefies or absorbs whatever mechanically adherent vapor may be present.

Description of the Campbell Apparatus for the Utilization of the Vapor of Ammonia in Engines.*

The plant embraces one horizontal return tubular boiler, set in brick work, one single cylinder Porter-Allen automatic cut-off engine, the apparatus for absorbing the exhaust vapor, and the various appliances which serve for equipment. These are arranged in the general manner shown in the accompanying ground plan of the works. (Plate XV.)

Boiler.

The boiler, in general features, is of the horizontal return tubular type, but it differs from those in common use in being provided with superheating surface. The shell is completely filled with tubes arranged in horizontal rows. In the back connection, a division plate is introduced, laid horizontally, so as to cause the products of combustion to pass forward through the tubes in the lower half of the shell, which are immersed, and then backward through the superheating tubes in the

* The following description of plant is taken, in part, for reason of its clearness, from the report of George H. Barrus, Esq., M. Am. Soc. Mech. Engrs., who made a number of tests upon the plant, the results of which are hereinafter given.

upper half of the shell. On arriving at the back connection above the plate, they enter the base of the smoke stack. The front head of the boiler is protected from the direct contact of hot gases by means of a covering of asbestos board. The front ends of the superheating tubes are likewise protected for a distance of some 12 inches by cylindrical shields made of sheet-iron, which are placed loosely in the tubes. The boiler is fitted with a dome, the vapor from which passes into a vertical drum 30 inches in diameter, 8 feet high, which rests on the front end of the setting and drains back into the boiler. This drum adds to the vapor-space of the boiler, and secures a larger proportion of vapor-space to liquor-space than is usually provided in steam plants.

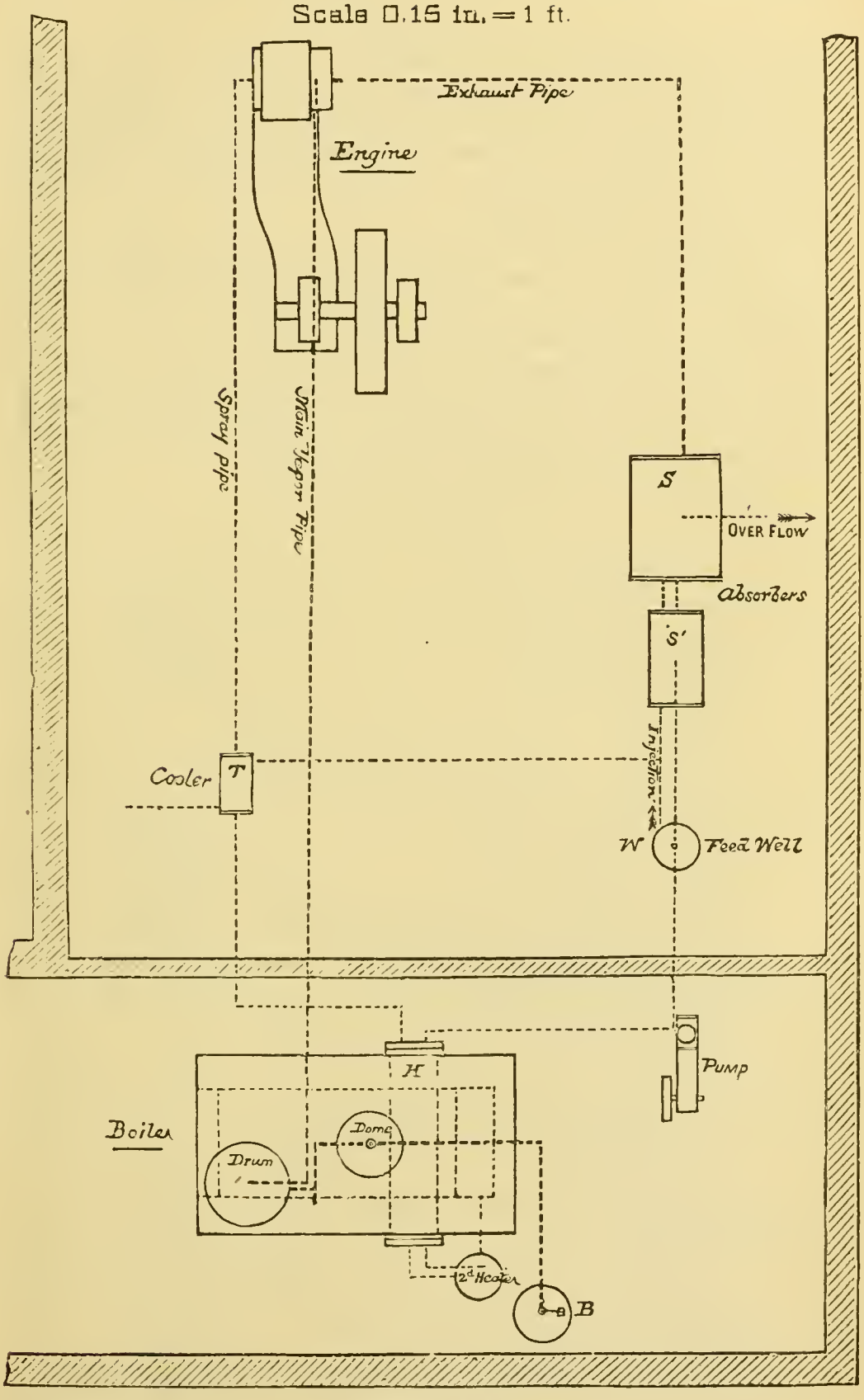
The connecting pipe is 4 inches inside diameter, and this is provided with a safety valve set at 110 pounds pressure, the vent of which is piped over into a tank (marked *B* on the plan), called the "dynamical reservoir," which is half full of aqua ammonia, the pipe descending to the bottom. A second safety valve set at 130 pounds pressure is provided, the vent of which passes to the atmosphere. The pipe which carries the vapor to the engine is attached to the top of the drum.

Owing to the insufficient natural draught of the smoke stack, the boiler is worked with artificial draught. For this purpose a small fan is provided (not shown on the plan), which is operated by power supplied by the engine. The discharge pipe of the fan connects with a brick passage leading from the back wall of the setting to the hollow bridge wall at the back end of the furnace, whence it passes into the ash pit through small openings at the bottom. The location of this passage, as also the general features of the setting of the boiler, are shown in the accompanying sketches. (Plates XV and XVI.)

The feed liquid is supplied to the boiler by means of a horizontal double acting fly-wheel pump, furnished with power from the engine. The cylinder is 3 inches in diameter, the stroke 9 inches, and the speed 70 revolutions per minute. The pump draws from the feed well and discharges through two heaters and a coil, the last-named being located in the back connection. The first heater (marked *H* on the plan), which is 11 inches in diameter and 7 feet long, is placed under the boiler behind the bridge wall, and buried in the setting. The feed liquor passes through a coil of ¾-inch pipe, 100 feet in length, which is wound in a spiral and placed within the cast-iron shell of the heater. The second heater, which is 12 inches in diameter and 4 feet high, is placed in a vertical position on the right-hand side of the boiler, outside the setting and near the back end. The feed liquor here passes through an additional length of 75 feet of ¾-inch pipe, put together with return bends. Surrounding the pipes in these heaters, is the "spray" as it is called, which is on its passage from the boiler to the liquefying apparatus of the exhaust vapor, and here the heat of the spray, which would otherwise be lost, is in part taken up. The feed liquor on leaving

Ground Plan showing general features of Plant.

Scale 0.15 in. = 1 ft.



the heaters passes through a coil containing 62 feet in length of 1-inch pipe, which is exposed to the products of combustion in the back connection below the division plate (not shown), and thence to the boiler.

Should the supply of liquor in the boiler become in any way reduced below the desired amount, it can be replenished from the "dynamical reservoir," and for this purpose a branch is carried to the reservoir from the suction pipe of the pump.

The vapor pipes are covered with hair felting laid outside of asbestos board, there being an air space between the board and the outside of the pipe. The dome of the boiler and the drum are covered with non-conducting cement, also laid outside of an air space.

The principal dimensions of the boiler, so far as they relate to the subject in hand, are given in Table No. 1. The figures for heating surface in the table do not include that of the coil in the back connection. The surface in this coil measures 21.4 square feet.

TABLE No. 1.

PRINCIPAL DIMENSIONS OF BOILER AND ENGINE.

BOILER—One Horizontal Return Tubular, set in brick work.

1	Diameter of shell	42 inches.
2	Length of shell	10 feet.
3	Number of 2-inch tubes below level of liquid	67.
4	Number of 2-inch tubes above level of liquid	68.
5	Inside diameter of tubes	1.75 inch.
6	Length of grate (net)	2.75 feet.
7	Width of grate	3.33 "
8	Width of metal and air spaces in grate	$\frac{5}{8}$-inch.
9	Area of water heating surface	369.3 square feet.
10	Area of steam heating surface	318.8 "
11	Area of grate surface	9.17 "
12	Collective area for draught through 67 tubes	1.12 "
13	Rated horse-power of boiler on basis of 15 square feet of water heating surface per horse-power	24.6 H. P.
14	Ratio of water heating surface to grate surface	40.3 to 1.
15	Ratio of steam heating surface to grate surface	33.6 to 1.
16	Ratio of grate surface to tube area	8.2 to 1.
17	Diameter of smoke stack	20 inches.
18	Height of smoke stack above grate	30 feet.

Engine.

The engine is of the Porter-Allen type. It is unjacketed, and is built in all essential respects in the manner customarily followed in engines of this kind. In these engines there are two steam valves and two exhaust valves. The steam valves consist of vertical slides, with faces on both the back and front sides. The back of the valve slides over the port. The front slides on a plate by which the valve is balanced, and this plate may be made to bear with greater or less force upon the valve by means of an adjusting bolt worked from the outside of the steam chest. It is needless to point out exactly how this adjustment is secured, further than to state that it is arranged so as to move the plate bodily toward the valve or away from it, as desired. These valves are operated by means of a link motion, the position of the block in which is varied by the action of the governor. The speed of the engine is controlled by this means and it is thus rendered an automatic cut-off engine. The exhaust valves are plain slides working in chests on the exhaust side of the cylinder. These chests and the spaces in the steam valves occupy so much volume that the clearance of the engine is made very large. The exact amount of clearance in this particular case has not been measured, but the assumed quantity of 10 per cent. corresponds to that of similar engines where it has been actually determined. The valve rods and piston rod of the engine are provided with metallic packing. One of the steam valve rods is kept cool by means of a stream of cold water. There is no oil cup attached to the cylinder and no lubricant is used in the cylinder other than that furnished by the ammonia. A small stream of liquid from the boiler is admitted while the engine is first starting, and at the time of stopping.

The principal dimensions of the engine, which relate to the subject in hand, are given in Table No. 2.

The load on the engine consists mainly of two Edison dynamo machines, class Y, having a nominal capacity of 225 amperes each. The speed of the dynamo shaft is 1 125 revolutions per minute when that of the engine is 205 revolutions per minute. The electrical plant is fitted with ampere meters, one for each dynamo, for showing the quantity of current delivered by the machines. The volt meter is not in use, but when the dynamos are working in normal condition, their electro-motive force is reported to be 93 volts.

The engine also drives the fan and feed pump which have been referred to, the pump which circulates the water in the absorbers, and a small amount of shafting.

TABLE No. 2.

ENGINE—Porter-Allen Automatic Cut-off, Single Cylinder.

19	Diameter of cylinder	11.5 inches.
20	Stroke of piston	20. "
21	Diameter of piston rod	1¾ "
22	Clearance (assumed)	10 per cent.
23	Horse-power constant 1 pound M. E. P., one revolution per minute	0.01037 horse-power.
24	Diameter of steam pipe	4 inches.

THE ABSORBING APPARATUS.

The absorbing process consists, first, in the introduction into the exhaust pipe of a jet of liquid from the boiler, called the "spray." The jet is ⅛ of an inch in diameter, and its point of entrance to the exhaust pipe is at the junction of the two branch pipes leaving the cylinder. The spray pipe is brought from the heaters, which have been referred to, and carried to a cooler (marked *T* on the plan), and thence to the exhaust pipe. The cooler consists of a coil of pipe enclosed in a vessel through which cold water circulates.* A branch from the outlet pipe of the cooler is carried to the feed well (*W*) for the use of the jet condenser, which has been referred to.

The exhaust vapor, after leaving the spray jet, passes to the first "absorber," (marked *S*), which is practically a surface condenser. This absorber has 500 tubes, ⅝-inch outside diameter, 41 inches long. The vapor then passes into the second absorber (*S'*), which is a surface condenser containing 150 tubes, ⅝-inch outside diameter, 40 inches long. Here the vapor is nearly all, if not wholly, converted to a liquid state, and the resulting liquid overflows into the well (*W*), from which it is returned to the boiler by means of the feed pump. The feed-well serves not only as a storage reservoir for the feed liquid, but also as a jet condenser, if necessary, for converting any vapor which remains into the liquid state.

The "cooler" and the "absorbers" are kept cool by means of a current of circulating water, part of which is supplied by the city hydrant, and part by a small centrifugal pump (not shown), which is fed with sea water drawn from North River, near where the works are located.

GENERAL CONDITION OF THE PLANT.

As far as can be judged by a somewhat careful examination, the plant appears externally to be in excellent working condition. An examina-

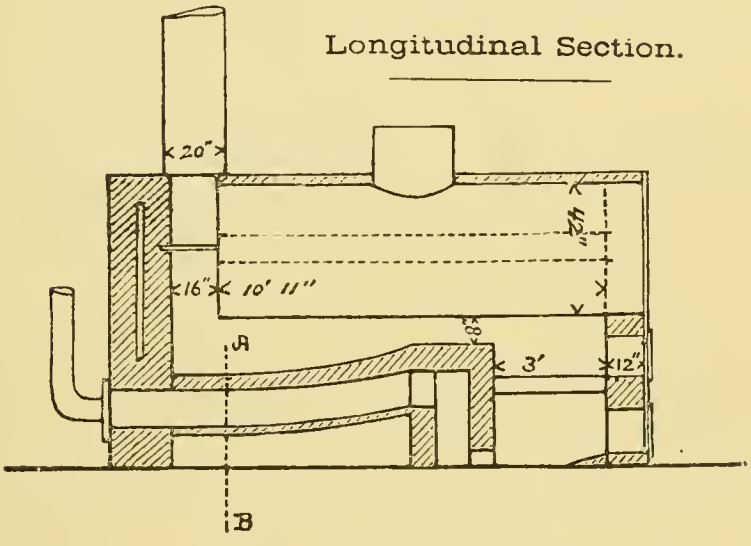
* This cooler has, subsequent to the tests, been found unnecessary, and has been dispensed with.

Sketches showing general features of

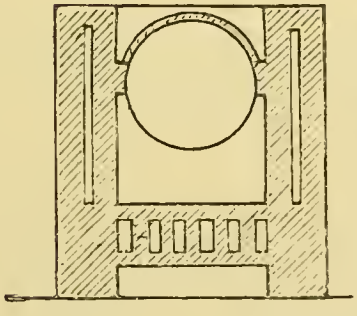
Boiler Setting.

Scale 0.15 in. = 1 ft.

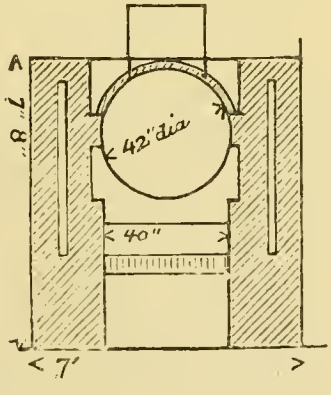
Longitudinal Section.



Section at A B.



Cross Section through furnace.



tion of the engine for leakage, however, revealed the fact that, in this particular, the plant is not in good condition. The valves and piston were tested for leakage when the engine was at rest, by applying the pressure of ammonia gas to them, and observing the amount of vapor which leaked through. The steam valves were tried by setting the rocker arm, which moves them, at mid-throw, in which position the ports are both covered, and opening the throttle-valve. The leakage was made to appear through the open indicator cocks. It was found that the valve at the crank end was nearly tight, but the valve at the head or front end leaked badly. Adjusting the balancing plate at this end produced no sensible improvement. The piston and exhaust valves were tested by placing the piston first at one end of the cylinder and then at the other end, and in each position opening the throttle-valve. In one position the piston and valve at one end were tested, and in the other position the piston and valve at the opposite end. The test consisted in admitting the full pressure of vapor, then shutting the throttle-valve and observing by means of repeated trials at the indicator cocks, the time when the vapor had all escaped from the cylinder. The vapor had all escaped from the front end of the cylinder at the expiration of two minutes from the time the throttle-valve was closed, and from the crank end at the expiration of only half a minute. This test does not show the exact location or amount of the leakage, but the writer draws the conclusion that the piston is fairly tight, and that both exhaust valves leak, the front-end valve a small amount, and the crank-end valve quite badly.

These remarks apply to the leakage at the time of the test of March 8th to 12th. Subsequently the crank-end steam valve became injured, and at the time of the tests of April 16th and 29th, although it had been repaired, it was still in an imperfect condition, as the leakage test revealed. Both steam valves on the last test leaked very badly. The exhaust valve and piston appeared to be in about the same condition as before.

The inside of the cylinder was examined by the writer, and found smooth, bright and free from corrosion. The steam valve at the front end was also examined, and both its faces, and the seats on the chest and balance plate, were likewise found in good condition.

The three leaks above mentioned operate against the economical working of the plant as follows:

First.—Steam leaks, (*a*) by raising the mean pressure during expansion above that required to accomplish its work and regulated by the engine through the cut-off. (*b*) Unnecessarily supporting the temperature during expansion, thereby not only wasting heat but also requiring more condensing water to cool the rejected fluid.

Second.—The exhaust leak, by raising exhaust temperature by means of unexpended heat in the fluid, which otherwise would, if imprisoned until release occurred in the regular way, yield its heat in accomplishing

work, but which, under the present conditions, demands an increased volume of condensing water for its reduction to the feeding temperature.

The present eccentric and link is not of the latest pattern of the Porter-Allen engine. The cards now exhibit the maximum possible point of release corresponding to the late compression. The more modern link, with the greater throw, permitting a later release with a later compression (thus approaching as nearly as is possible under the existing conditions of speed a card of the so-called Corliss type), would, in the writer's opinion, show an even more economical result, for the reason that the initial temperature of the fluid being considerably less than that of steam at a corresponding pressure, the initial condensation and subsequent re-evaporation would necessarily be less; and, therefore, from a thermodynamical point of view, the compression should be less than in an engine using steam as a working fluid, and consequently the work done in compressing could be reduced in a like ratio, thus increasing the efficiency of the machine.

The Results of the Experiments.

The experiments were conducted on a comparative basis, *i. e.*, the plant was operated with steam and with ammonia, the same observations being taken in each case; the results are given in the following table:

	1	2	3	4	5
Number of test	1	2	3	4	5
Date of test	March 8th	March 9th	April 16th	April 29th	March 12th
Experimenter	Barrus	Barrus	Barrus	Magovern	Barrus
Vapor used	Ammonia	Ammonia	Ammonia	Ammonia	Steam
Fuel used	Anthracite	Anthracite	Anthracite	Anthracite	Anthracite
Duration of tests, hours	8	10	7.45	10	8.005
Weight of coal consumed, pounds	1 400	1 400	1 098	1 550	1 453
Weight of ashes, pounds	.....	139	90	68	138
Per cent. of ashes	.....	9.9	8.2	4.38	9.5
Coal consumed per hour, pounds	175	140	147.4	155	181.5
Coal consumed per square foot of grate per hour, pounds	19.09	15.27	16.07	16.90	19.98
Injection water per hour, pounds	.....	.....	372.8	500.1	.....
Average gauge pressure, pounds	100	95.5	86.6	89.45	66.5
Temperature at throttle	.....	304°	301°	324°	.....
Temperature of exhaust	.....	.....	180.8°	180 5°	.....
" spray	.....	280°	271°	279°	.....
" feed	.....	167.6°	167°	173°	.....
" uptake	.....	390°	394°	416°	394°
" Croton	.....	.....	49.5°	.....	.....
" sea water	.....	.....	42.7°	46.6°	.....
" discharge	.....	.....	104.2°	92.5°	.....
Vacuum, inches	.....	11.5	11	7	.....
Revolutions	205.2	204.5	201.5	200.14	198.3
Mean efficiency, pressure	29.05	27.13	26.99	28.49	15.7
Indicated horse-power	61.8	57.53	54.00	59.25	32.26
Coal consumed per horse-power per hour	2.832	2.433	2.729	2.616	5 626

Average 2.652

An inspection of the above tables will show:

First.—That with a boiler capable of supplying an engine with steam but for 32.26 horse-power, it was found capable of furnishing to the same engine, when ammonia was substituted for steam, sufficient vapor for developing 58.14 horse-power at a lower rate of combustion, thus furnishing an increased potential capacity of over 80 per cent.

Second.—That in an engine of modern type, requiring 5.626 pounds of coal per horse-power per hour when operated with steam, the consumption fell 2.974 pounds of coal per horse-power per hour, or over 53 per cent.

It may be urged by some that a portion of this economy may be due to the engine being practically a condensing engine when ammonia is used. The writer is not inclined to accept this view of accounting for the economy.

First.—Because the cards show but little vacuum in the cylinder, the value of which, when figured, will be found to be less than 10 per cent.

Second.—Because no vacuum pump is employed.

Third.—Because there has yet to be called to our attention an engine whose efficiency, by virtue of conversion from non-condensing to condensing, has been increased over 50 per cent. economically and 80 per cent. potentially.

The performance of the Porter-Allen engine in this case was not what might have been expected from so good an engine. The writer attributes it to two causes:

First.—To the load being lighter than that of maximum economy.

Second.—To the low boiler performance, being but 7.864 pounds of water per pound of coal under actual boiler conditions.

The engine, with a good boiler and run to its rated capacity, would be capable, in the writer's judgment, of developing a horse-power, with steam, for the expenditure of about 4½ pounds of coal per hour.

The poor boiler performance affected the results of both fluids equally; the low power of the steam trial affected the economical generation of power slightly more in the steam than in the ammonia test, though how much would be difficult to say.

The discrepancy between the theoretical and actual performances of aqua ammonia, as before intimated, must not be overlooked. The writer attributes the discrepancy to the phenomena that Faraday and others have observed, viz., that at all temperatures above freezing, water gives off vapor, though it is not completely vaporized until it attains a temperature of boiling dependent upon the pressure.

The writer believes that considerable water is vaporized with the liberation of the ammonia, and cites in evidence of this belief, the large amount of heat rejected in the circulating water shown by its quantity and rise in temperature.

INDICATOR CARDS, TAKEN WITH TABOR INDICATOR FROM A PORTER-ALLEN ENGINE, USING AMMONIA GAS IN THE CYLINDER.

SCALE OF CARD 40.

TIME 1:45 P. M.

Average revolutions per minute 200.14.

Boiler Pressure 90 lbs.

Vacuum in Feed Well 7 in.

Back End.

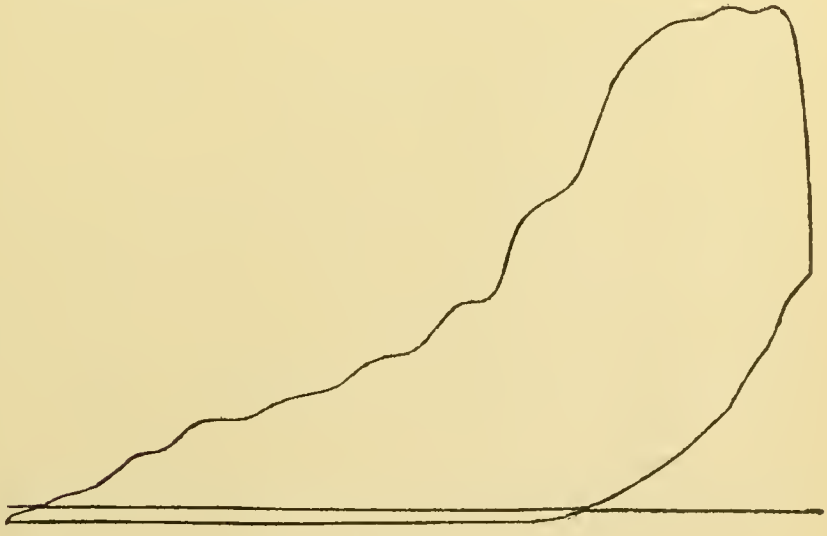
SCALE OF CARD 40.

TIME 1:45 P. M.

Average revolutions per minute 200.14.

Boiler Pressure 90 lbs.

Vacuum in Feed Well 7 in.



Crosshead End.

Vol. XIX, p. 145.

PLATE XVII.

The steam vapor present, though undesirable from an economical point of view, is desirable in a practical way by reason of its assistance in the absorption process.

In conclusion, the writer desires to reiterate, and believes he has demonstrated in both a theoretical and practical way, the statement that a calculation of the efficiency of an engine, save of the elementary type, by the formula $\frac{T_1 - T_2}{T_1}$ is incomplete, unless its method of heat reception is, like its expenditure, independent of the nature of the fluid employed; if the fluid is supplied as a permanent gas, such as air, hydrogen, etc., the Rankine formula can properly be applied; where, however, different liquids, with varying latent heats are employed, the $\frac{T_1 - T_2}{T_1}$ formula is insufficient, as it involves no consideration of the varying latent heats which must be supplied by the expenditure of fuel, and are therefore part and parcel of a consideration of the efficiency of the system.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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(Vol. XIX.—October, 1888.)

HIGH WALLS OR DAMS TO RESIST THE PRESSURE OF WATER.

By JAMES B. FRANCIS, Past President Am. Soc. C. E.

READ MAY 16TH, 1888.

WITH DISCUSSION.

In recent investigations, principally by French engineers, the crushing strength of the material of which the wall is constructed is made a controlling factor. No objection could be made to this if the numerical values of the resistance to crushing were determined under the same conditions as in the cases to which they are applied. The mortar is usually weaker than the stone. If made of Portland cement and thoroughly set, its resistance to crushing, as deduced from hand specimens, may be taken at a ton per square inch. Experienced engineers adopt one-tenth of the breaking weight as the safe load in ordinary masonry, to be reduced to a fifteenth or twentieth in rubble masonry.* One-tenth of a ton per square inch is equal to 200 pounds per square inch, and

* Poncelet, Traite de Mécanique Industrielle, 2d Edition, Vol. I, Sect. 264.

this is about the maximum pressure to which the masonry is subjected in any of the high reservoir walls or dams that have been constructed, so far as we have accounts of them.

Are we justified in applying the results of experiments on the resistance to crushing made on small specimens to large masses of the same material? I think not. Two-inch cubes of the strongest granite are crushed with a load of about 12 tons per square inch, equivalent to a column of the same material 1 inch square and nearly 4 miles high. If the results of the experiments on the 2-inch cubes would apply, a granite formation would be crushed at this depth; and granite being the strongest of rocks, at this and all greater depths, the materials that are solid near the surface of the earth must all be crushed. In the absence of proof that this is the condition, in the light of experience it will be admitted that it is not so: but, on the contrary, we should expect to find granite and all other rocks at that depth to be at least as sound as at any less depth.

What are the differences in the conditions in the two cases?

Any part of the interior of a great mass of uniform material, and subject to uniform pressure, is prevented from yielding laterally by the adjoining parts, which are under the same pressure, and react with equal force; while in small masses the lateral support is slight. The material in the interior of the great mass is compressed into a somewhat smaller volume than it would be under less pressure, but within its original horizontal dimensions, and is not fractured, as that would require a lateral movement of some of its parts, which cannot take place.

Somewhat similar effects have been observed in hand specimens of different sizes subjected to crushing pressures. We are indebted to General Gillmore* for valuable experiments on this and other points. He found that the resistance to crushing per square inch of surface increased rapidly with the area of the bed subjected to the pressure. For instance, he found that a 1-inch cube of yellowish-gray Berea sandstone was crushed with a pressure of 7 000 pounds. Four of these placed side by side would, therefore, require 28 000 pounds to crush them; while a block 2 inches square and 1-inch thick, having, of course, the same area of bed as the four 1-inch cubes, had a compressive strength of 38 000

* Official report to the Chief of Engineers of the United States Army, on the compressive strength, specific gravity and ratio of absorption of the building stones in the United States, by Q. A. Gillmore, Lieutenant-Colonel of Engineers, Brevet Major-General, U. S. A., dated August 10th, 1875.

pounds, or about thirty-six per cent. greater than the sum of the compressive strength of the four 1-inch cubes. In the same report he gives, in detail, the results of experiments showing the increase in resistance to crushing per square inch of cubes of two varieties of Berea sandstone as the size of the cube is increased. He found that his average results are given very closely by the formula $y = C\sqrt[3]{x}$, in which $y =$ the average compressive strength in pounds per square inch of surface, and $x =$ the side of the cube in inches. The following tables, Nos. 1 and 2, contain summaries of the results given in General Gillmore's Tables, Nos. 1, 1a, 2, 2a, to which I have added in the final columns, approximate estimates of the resistance to crushing at the middle of the surfaces, which I arrive at as follows:

In the comparison of the compressive strength of a 1-inch cube and a block 2 inches square, the thickness was the same, and the increased strength was evidently due to the increased area. The parts near the sides of the cube and block must have been of substantially the same strength per square inch, and the increased strength of the block must have been mainly in its central parts, and the maximum strength near the middle. Applying the same reasoning to the experiments on the sandstone cubes in Tables Nos. 1 and 2, I assume that the resistance per square inch of surface at the edges of the cube is very small, and at the middle it is a maximum. If the resistance per square inch of surface to crushing increases uniformly from the edge of the cube to its middle point, the total pressure on the surface may be represented by a pyramid. Let *A B C D*, Fig. 1, Plate XVIII, be a plan of the cube surmounted by the pyramid, and *E F G* be a vertical section of the pyramid through *H I* and *K L*, and let *E F N M* be a prism whose base is the surface of the cube, and *E M* and *F N* the average resistance per square inch to crushing, then the volume of the prism will represent the total resistance of the cube to crushing; and in order that the volume of the pyramid may have the same volume as the prism and also represent the total resistance of the cube to crushing, its height *O G* must be three times the height *O P* of the prism; that is, the resistance per square inch to crushing at the middle of the cube, where it is a maximum, is three times the average resistance per square inch of the whole surface.

It will, however, be more rational to assume that the law of the increase of resistance per square inch from the edges to the middle is

represented by a curved line. The data furnished by General Gillmore's experiments may be sufficient to determine its precise form, but if so, I have not been able to find it. Some of its characteristics are, however, obvious. It must be symmetrical about the axis $O\ G$, the maximum ordinate being on this axis. The ordinates at E and F, corresponding to the edges of the cube, must be small, and a tangent at the vertex must be perpendicular to the axis. Two curves with these characteristics are given at Fig. 1, Plate XVIII. The curve $E\ Q\ F$ is a parabola, and the curve $E\ R\ F$ is an ellipse. The solid on the square base, of which the parabola is a section through $H\ I$ and $K\ L$, may be called a paraboloid with a square base, and in order that its volume may be the same as that of the prism $E\ F\ N\ M$ its height $O\ Q$ must be double the height of the prism; that is, the maximum resistance per square inch to crushing is twice the average resistance per square inch of the whole surface.

Similarly for the curve $E\ R\ F$ the solid may be called a semi-prolate spheroid with a square base, and in order that its volume may be equal to that of the prism, its height $O\ R$ must be one and a half times the height of the prism; that is, the maximum resistance per square inch to crushing is one and a half times the average resistance per square inch of the whole surface.

Either of these results would be a sufficient approximation for the present purpose, which is simply to show that the resistance to crushing is much greater at the central parts of a cube of homogeneous material than near the sides.

The estimates of the compressive strength at the middle of the cube given in Tables Nos. 1 and 2, are from the paraboloid, which I take as giving medium results, and according to which the strength at that point is twice the average strength of the whole cube.

In Mr. John Grant's paper on the strength of Portland cement, in the Proceedings of the Institution of Civil Engineers, read December 12th, 1866, he gives the results of numerous experiments on the resistance to crushing of blocks of neat Portland cement, $3 \times 3 \times 1\frac{1}{2}$ inches, of different ages, crushed in a hydraulic press, part flat and part on edge. Ten blocks of each age and in each position were tested.

The following is a summary of the results:

Eighty blocks crushed on edge.

Area exposed to pressure, $3 \times 1\frac{1}{2}$ inches $= 4.5$ square inches.

Average age, in water, seven days; in air, six hundred and ninety-nine days.

Average tensile strength per square inch, 303 pounds.

Average compressive strength per square inch, 4 956 pounds.

Compressive strength $\frac{4956}{303} = 16.36$ times the tensile strength.

Seventy blocks crushed flat.

Area exposed to pressure, 3×3 inches $= 9$ square inches.

Average age, in water, seven days; in air, six hundred and sixty-six days.

Average tensile strength per square inch, 327 pounds.

Average compressive strength per square inch, 5 882 pounds.

Compressive strength $\frac{5882}{327} = 17.99$ times the tensile strength.

Comparing the two results, the blocks crushed flat have double the area exposed to pressure and one-half the thickness of the blocks crushed on edge, and have a resistance to compression per square inch about 11 per cent. greater.

In General Gillmore's experiments, the resistance per square inch to crushing cubes of yellowish gray Berea sandstone for the area of 9 square inches is about 12 per cent. greater than for 4.5 square inches area ; the two results indicating nearly the same increase of strength per square inch with the same increase of area exposed to pressure.

TABLE No. 1.

SHOWING the increase in resistance, per square inch, of cubes of yellowish gray Berea sandstone as the size of the cube is increased. These were pressed upon by wooden cushion blocks, ranging from one-sixteenth to a little over three-eighths of an inch in thickness, as the cubes increased, and of the same dimensions as the stone. They were all broken on bed.

Side of cube. Inches.	Number of cubes tested.	Average compressive strength per square inch. Pounds.	Compressive strength computed by the formula, $y=7\,000\sqrt[3]{x}$ Pounds per sq. in.	Estimate of the compressive strength at the middle of the cube. Pounds per sq. in.
0.25	8	4 992	4 410	8 820
0.50	10	6 080	5 558	11 116
0.75	7	6 347	6 356	12 712
1.00	9	6 990	7 000	14 000
1.25	9	7 342	7 539	15 078
1.50	10	8 226	8 015	16 030
1.75	9	9 310	8 435	16 870
2.00	9	8 955	8 820	17 640
2.25	7	9 130	9 170	18 340
2.50	10	8 856	9 499	18 998
2.75	10	9 838	9 807	19 614
3.00	3	10 125	10 095	20 190
4.00	1	11 720	11 112	22 224

TABLE No. 2.

SHOWING the increase in resistance, per square inch, of cubes of blue Berea sandstone as the size of the cube is increased. These were pressed upon by hardened steel surfaces. They were all broken on bed.

Side of cube. Inches.	Number of cubes tested.	Average compressive strength per square inch. Pounds.	Compressive strength computed by the formula, $y=9\,500\sqrt[3]{x}$ Pounds per sq. in.	Estimate of the compressive strength at the middle of the cube. Pounds per sq. in.
1.00	11	9 500	9 500	19 000
1.25	10	10 300	10 231	20 462
1.50	8	10 730	10 877	21 754
1.75	9	11 377	11 447	22 894
2.00	10	12 000	11 970	23 940
2.25	10	12 500	12 445	24 890
2.50	9	10 635	12 895	25 790
2.75	7	13 200	13 309	26 618

In the experiments on crushing the cubes of sandstone and blocks of cement with a hydraulic press, it would seem that an equal pressure is applied to each square inch of the surface, and it evidently would be so, if applied to a perfectly rigid mass. All substances, however, so far as known, are compressible in some degree, stone and cement not being exceptions. Under the conditions in which the pressure is applied in these experiments, the amount of compression must be the same at each square inch of surface, but the resistance to compression is much greater at the central parts than near the edges. In other words, the pressure must be greater to produce the same amount of compression at the central parts than near the edges. The cause of this is, evidently, the less freedom of the central parts to yield laterally to the pressure, and the practical effect is to transfer part of the pressure from the parts near the edges to the central parts.

H. Gollner * reports some experiments made at Prague on the crushing strength of cylinders and cubes of Portland cement. The average

* Experiments on the strength of cements and on the bending of bars, by H. Gollner. Abstracts of papers in Foreign Transactions and Periodicals, in Proceedings of the Institution of Civil Engineers, 1877–78.

crushing strength per square inch of the cubes was about seventy-two per cent. of that of the cylinders. Fig. 2, Plate XVIII, represents the surface of the two blocks to which the pressure is applied. Taking the area of the inscribed circle as the unit, the area of the square is 1.273, and the total pressure upon it is $1.273 \times 0.72 = 0.917$, the total pressure on the cylinder being 1.000. This indicates that the angles of the cube, projecting beyond the cylinder, added nothing to the strength of the block, and that practically the pressure applied to them was transferred to the inscribed circle.

The preceding experimental results, and the deductions therefrom, all relate to hand specimens of stone and hardened cement, of regular prismatic forms, subjected to external pressures; but the deductions therefrom, I believe, may be extended to great masses of any form, subjected to gravity or external force, or a combination of the two; the fundamental principle being, that different parts of the mass yield unequally to pressure, depending on the opportunities to yield laterally to the direction of the pressure, the effect being to transfer pressure from the parts yielding more easily to compression to those yielding less easily.

The transfer of pressure takes place in tunnelling through rock, chalk and clay, in cases where the excavation does not require timbering to support the roof before the lining of masonry is completed.

The tunnels under Lake Michigan, at Chicago, were constructed for the purpose of drawing water from the lake for the supply of the city, at a sufficient distance from the shore to be uncontaminated by sewage. They each extend about two miles under the lake. The formation is clay, with occasional pockets and beds of sand of small extent, and some bowlders. The tunnels are mostly through clay, in which the roof did not need support before the masonry was put in.

Fig. 3, Plate XVIII, is a cross section through both tunnels, where the depth of water over them was the greatest. They are both substantially at the same level, their centers being about 65 feet below the surface of the lake, and they are about 50 feet apart. The smaller, or old tunnel, is intended to supply fifty millions of gallons of water in twenty-four hours, and was completed in 1867; it is five feet in diameter inside the lining, which is of brick, and 8 inches thick. The larger, or new tunnel, is intended to supply one hundred millions of gallons in

twenty-four hours, and is seven feet in diameter inside the brick lining, which is one foot thick. It was completed in 1874.

The excavation was "generally through stiff blue clay. * * * It very seldom required bracing, when not left to support itself more than thirty-six hours,"* Professor Andrews describes it "as a solid, impermeable clay."† In the new tunnel the progress was usually about 10 feet per day. The work was carried on by eight-hour shifts. In the first shift the main part of the excavation was taken out; the second shift completed the excavation, trimming it to the exact size and form to receive the lining; the third shift put in the brick lining, which is built up solid to the clay.

The depths of water and clay at the part represented in the section, Fig. 3, Plate XVIII, were about equal. The pressure at the level of the center of the tunnels, before any excavation was made, was

Water	$32.5 \times 62.5 = 2031.25$	pounds.
Clay	$32.5 \times 120. = 3900.00$	"
Total	5931.25	"

or 2.965 tons per square foot of area; and this pressure was, of course, uniform over an area of indefinite extent in all directions. When the excavation was made for the new tunnel, the support for 9 feet in width being taken away, an additional pressure of $9 \times 2.965 = 26.685$ tons per foot in length of the tunnel was transferred to adjacent areas; and not only this, but the pressure on the areas adjoining the excavation on each side was partially transferred to more distant areas, otherwise the clay on the sides of the excavation would be forced in. No yielding of the roof or sides was observed, but it must have taken place, in a slight degree, during the progress of the excavation, in order that the pressure might be transferred. In such cases it is commonly said that an arch is formed in the superincumbent material, which removes the pressure from immediately beneath it and transfers it to the sides. This explanation, of course, involves a small yielding of the material, in order to admit of a new adjustment of the pressures. I think a more rational explanation is, that a beam is formed, carrying a load uniformly distributed and resting on a considerable area on

* First annual report of Department of Public Works of the City of Chicago, for the year ending December 31st, 1876.

† The American *Journal of Science* (Silliman's Journal), Second Series, Vol. XLIII.

each side of the excavation. The beam deflects under the load an amount corresponding to the compression of the material supporting it, and rather more immediately over the excavation. The roof of the excavation is held up by the adhesion of the material to the under side of the beam.

This explanation is represented by the dotted lines in Fig. 3, Plate XVIII, a little above the tunnels; the straight lines *A B*, representing the bottom of the beam before the excavation is made, and the curved line, the bottom of the beam after the excavation is made, the vertical scale being greatly exaggerated. The pressures on the sides of the excavations must be entirely removed, as moist clay in that situation would not carry any sensible load. Similar compressions and transfers of pressure must take place in rock and chalk similarly situated; the magnitude of the movements being inversely as the rigidity of the material.

I propose to apply the preceding deductions to high walls and dams of masonry designed to withstand the pressure of water. I assume that they are constructed of rubble masonry, constituting substantially a homogeneous mass; and I assume, in accordance with preceding experiments and deductions, that it is compressible to the extent of causing the transfer of the part of the pressures, which would otherwise be excessive near the exterior of the section, to the central part of the mass, where the resistance to crushing is much greater. This renders it unnecessary to take into account the crushing effect of the pressures, which we must do if the masonry is assumed to be perfectly rigid.

In determining the section of a wall to resist the pressure of water, I assume that it acts simply by its weight, and that it is built on a rock foundation, and that each foot in length has the necessary stability, independently of its connection with other parts of the wall.

The wall may fail from either of the following causes:

First.—It may be overturned on its base or on some higher joint in the masonry.

Second.—The wall may yield to the pressure by sliding on its base or at some horizontal joint in the masonry.

Third.—The material of which it is composed may be crushed.

As to the first-named cause, the mass of masonry required for stability depends on its weight per cubic foot, and upon the mass and form of the section.

The weight of rubble masonry may be taken as varying from two and one quarter to two and one half times the weight of water, depending on the nature of the materials. The stone being heavier than the mortar, the larger the proportion of stone, the heavier the masonry per cubic foot. If it contains a large proportion of large stone, it may be taken at two and one half times the weight of water, or at $62.5 \times 2.5 = 156.25$ pounds per cubic foot. If mostly of small stone, at $62.5 \times 2.25 = 140.62$ pounds per cubic foot.

On a horizontal base, the form of section having the required resistance to overturning, with the least amount of masonry, is a right-angled triangle. In Fig. 4, Plate XIX, $A\ C\ T$ is such a triangle, the apex A being at the level of the greatest height of water to be provided for. The up-stream side, $A\ C$, opposed to the pressure of the water, is vertical, and I call it the up-stream face, the angle C being the up-stream toe. The hypothenuse $A\ T$, I call the down-stream face, the angle T being the down-stream toe.

In Fig. 4, Plate XIX, let $A\ C\ T$ be the section, in which the resistance to overturning, on the toe T, is just equal to the pressure of the water on the hypothesis that the wall is a perfectly rigid mass, the level of the water on its up-stream side being at the level of the apex A of the triangle.

Put h = the height, $A\ C$.

x = the width of the base, $C\ T$.

w = the weight of a cubic foot of the masonry, that of water being unity.

The weight of the masonry is $\frac{h\ x\ w}{2}$. (1)

Its center of gravity is at g, such that the line $g\ B$ being vertical, $B\ T = \frac{2}{3}$ of $C\ T = \frac{2}{3}x$. Then the moment of the weight of the masonry to overturning on the toe T is

$$\frac{h\ x\ w}{2} \times \frac{2}{3}\ x = \frac{h\ x^2\ w}{3}. \qquad (2)$$

The pressure of the water against the up-stream face $A\ C$, is $\frac{h^2}{2}$, which is to be taken as acting at the center of pressure D, the height above the base, $C\ D$, being $\frac{h}{3}$. Then the moment of the pressure to overturning on the toe T is,

$$\frac{h^2}{2} \times \frac{h}{3} = \frac{h^3}{6}. \tag{3}$$

Equating the moments :

$$\frac{h\, x^2\, w}{3} = \frac{h^3}{6}$$

from which we find :

$$x = h \sqrt{\frac{1}{2\, w}} \tag{4}$$

Taking $w = 2.25$, then $x = 0.4714\ h$.

$w = 2.50$, then $x = 0.4472\ h$.

When $h = 100$ feet, and $w = 2.25$, numerically, the weight of the masonry is $\frac{2.25 \times 62.5 \times \overline{100}^2 \times 0.4714}{2} = 331\,453$ pounds, and the pressure of the water is $\frac{\overline{100}^2 \times 62.5}{2} = 312\,500$ pounds.

In Fig. 4, Plate XIX, g being the center of gravity of the masonry, construct the parallelogram $g\ E\ F\ G$, making $g\ E = 312\,500$ and $g\ G = 331\,453$, and draw the resultant $g\ F$, which intersects the base of the wall at T, where the total pressure falls, and the pressure per square inch is infinite.

Such a wall would have no stability, even if the material had perfect strength and rigidity, which, of course, it could not have.

To give the necessary stability, instead of making the resultant of the weight and pressure intersect the base at the toe T, it is the practice to make the base of such width that the resultant would fall within its middle third. Assuming it to intersect the base at one-third of its width from the down-stream toe, and calling the width x', the weight of the masonry is $= \frac{h\, x'\, w}{2}$. (5)

Fig. 5, Plate XIX, the center of gravity of the masonry being at g, its horizontal distance from the axis of moment is $\frac{x'}{3}$, and the moment of the weight of the masonry is $\frac{h\, x'\, w}{2} \times \frac{x'}{3} = \frac{h\, x'^2\, w}{6}$. (6)

Equating this to the moment of the pressure of water, which is the same as before, we have $\frac{h\, x'^2\, w}{6} = \frac{h^3}{6}$,

from which we find $x' = h \sqrt{\frac{1}{w}}$. (7)

Taking $w = 2.25$, we have $x' = 0.6666\ h$.

Taking $w = 2.50$, we have $x' = 0.6325\ h$.

When $w = 2.25$, the weight of the masonry for a wall 100 feet high, is

$$\frac{100^2 \times 0.6666 \times 2.25 \times 62.5}{2} = 468\ 703 \text{ pounds.}$$

The pressure of water being as before = 312 500 pounds.

In Fig. 5, Plate XIX.

$$C\ T = x' = 0.6666\ h = 66.66 \text{ feet.}$$

$$T\ H = H\ B = B\ C = \frac{x'}{3}.$$

In the parallelogram $g\ E\ F\ G$, make $g\ E = 312\ 500$, and $E\ F = 468\ 703$, and draw the resultant $g\ F$, which intersects the base $C\ T$ at H.

Most ledges of rock have seams in various directions which admit the passage of water. If such is the case with the rock on which the wall we are considering is built, we must assume that the water in the seams is in communication with the water in the reservoir, and that near C, Fig. 6, Plate XIX, is an upward pressure on the masonry equal to the head $A\ C$. The upward pressure may also extend over the whole base of the dam, naturally diminishing towards the toe T, where it would be zero. Supposing it to diminish uniformly from C to T, the average would be equal to that due to half the head $A\ C$, and the total upward pressure would be $\frac{h\ x''}{2}$, x'' being the width of the base $C\ T$. The center of the upward pressure will be at B, one-third of the width of the base from C, and equal to $B\ H$ and $H\ T$. The moment of the same on H is

$$\frac{h\ x''}{2} \times \frac{x''}{3} = \frac{h\ x''^2}{6} \qquad (8)$$

adding this to the moment of the horizontal pressure, which is $\frac{h^3}{6}$, (9)

we have for the sum of the moments of water pressures,

$$\frac{h^3}{6} + \frac{h\ x''^2}{6}. \qquad (10)$$

The weight of the masonry is $\frac{h\ x''\ w}{2}$, and its moment on H is

$$\frac{h\ x''\ w}{2} \times \frac{x''}{3} = \frac{h\ x''^2\ w}{6}. \qquad (11)$$

Equating the moments, $\frac{h^3}{6} + \frac{h\,x''^{\,2}}{6} = \frac{h\,x''^{\,2}\,w}{6}$.

From which we deduce $x'' = h\sqrt{\frac{1}{w-1}}$. (12)

Putting $w = 2.25$, we have $x'' = 0.8944\ h$.

Putting $w = 2.50$, we have $x'' = 0.8165\ h$.

When $w = 2.25$, for a wall 100 feet high, the weight of masonry is,

$$\frac{\overline{100}^{\,2} \times 0.8944 \times 2.25 \times 62.5}{2} = 628\,875 \text{ pounds.}$$

Deduct upward pressure of water

$$\frac{\overline{100}^{\,2} \times 0.8944 \times 62.5}{2} = 279\,500 \text{ pounds.}$$

Effective weight of masonry, 349 375 pounds.

The horizontal pressure of the water is the same as before, viz., 312 500 pounds.

Construct the parallelogram $g\ E\ F\ G$ and draw the resultant $g\ F$, which intersects the base at H, at one-third the width of the base from the toe T.

In computing the required dimensions of the parts of bridges and other structures of wood and iron, to sustain permanently given strains, factors of safety are introduced, usually of not less than four; that is to say, the computed strength is four or more times the assumed breaking strain. This is needed, not only to give stability from having a surplus of strength, but in order to provide for the uncertainty as to the original strength of the materials, and also for the depreciation in its strength, which ordinarily takes place from age and long-continued use. In a wall, to sustain the pressure of water, we can and ought to assume a height of water and pressure that cannot be exceeded. The wall, which is assumed to act simply by gravity, has no uncertainty as to its weight, and is unchangeable in its power of resistance, and a much less factor of safety is accordingly needed than in constructions of iron and wood.

The section in Fig. 4, Plate XIX, computed by formula (4) has no factor of safety. The sections in Figs. 5 and 6, Plate XIX, computed by formulas (7) and (12) have factors of safety of two; that is, the weight of the masonry is double that required to resist overturning on the toe T, from the pressure of the water. In the parallelograms of forces in Figs. 5 and 6, $g\ G$ and $E\ F$ represent the weight of the

masonry; if this is taken at one-half its real value, the corresponding parallelograms will be $g\ E\ i\ k$, and the resultants will pass through the toe T, as it does in Fig. 4.

Generally the width of base, when there is no upward pressure of water, for any factor of safety, f, is given by the formula

$$x' = h\sqrt{\frac{f}{2\ w}} \tag{13}$$

and when there is an upward pressure, as taken above,

$$x'' = h\sqrt{\frac{f}{2\ (w-1)}}. \tag{14}$$

As to the second-named possible cause of failure of the wall, viz., that it may yield to the pressure by sliding on its base, or on some horizontal joint in the masonry.

In the section of wall represented by Fig. 5, Plate XIX, the weight of masonry above the base or any joint, h feet below the apex of the triangle, is $\frac{62.5\ h\ x'\ w}{2}$; by formula (7), $x' = h\sqrt{\frac{1}{w}}$; substituting the value of x', we have for the weight, $\frac{62.5\ h^2\ \sqrt{w}}{2}$.

The pressure of water against the wall for the same height h, is $\frac{62.5\ h^2}{2}$; consequently the weight of the masonry is equal to the pressure against it, multiplied by $\sqrt{w}$.

Taking $w = 2.25$, $\sqrt{w} = 1.5$; that is, the weight of the wall is 1.5 times the pressure against it, and in order to slide, the co-efficient of friction must be less than $\frac{1}{1.5}$, or than 67 per cent.

The experiments of Morin indicate that the co-efficient of friction for Oolitic limestone upon Oolitic limestone, the surfaces in contact being smooth planes, is 55 to 75 per cent. Weisbach gives for the "co-efficient of friction of rest," for stone or brick upon stone or brick, well polished, 67 to 75 per cent.

The wall we are considering is founded upon a surface of rock, which, if not naturally uneven, should be made so. The wall is of rubble masonry, laid in hydraulic mortar, but not in courses. There can therefore be no smooth planes without cutting through parts of the rock or some of the rubble stones. Omitting the cohesion of the mortar, the co-efficient of friction must be many times the superincumbent weight,

instead of two-thirds or three-quarters, as required in order to slide. We may therefore conclude that there is no danger of failure from this cause.

As to the third cause from which the wall may fall, viz., the material of which it is composed may be crushed.

According to the method given by Debauve,* when the reservoir is full, and the resultant of the pressure of the water and of the weight of the masonry intersects the base at one-third of its width from the down-stream toe, the maximum pressure per square inch on the base is at the down-stream toe, and is double what the pressure per square inch would be if the weight was uniformly distributed over the whole width of the base.

When the reservoir is empty the conditions are reversed; the maximum pressure being at the up-steam toe, and equal to double the average pressure on the base of the wall.

The weight of the masonry for any height of wall h, by formula (5) is $\frac{h\,x'\,w}{2}$. Then the average pressure per square inch on the base for $w = 2.25$ is $\frac{62.5 \times 2.25\,h}{2 \times 144} = 0.4883\,h$, pounds,

and the maximum pressure, per square inch, according to Debauve, is

$$2 \times 0.4883\,h = 0.9766\,h, \text{ pounds.}$$

The pressure and the width of the base by formula (7), for several heights of wall, are given in the following table:

TABLE No. 3.

Height of Wall. h Feet.	Pressure, in pounds per square inch, on base.		Width of base x' for $w = 2.25$. Feet.
	Average.	Max. according to Debauve's method.	
100	48.83	97.66	66.66
200	97.66	195.32	133.32
300	146.49	292.98	199.98
400	195.32	390.64	266.64
500	244.15	488.30	333.30

* "Dams and Reservoirs," translated by E. S. Gould from *Manuel de L'Ingenieur des Ponts et Chausees*, by A. Debauve, New York, 1881.

In Fig. 7, Plate XX, the section $A\ C\ T$ is drawn according to formula (7). $T\ S$ represents the pressure at the down-stream toe T, when the reservoir is full; according to the method given by Debauve, the pressure at the up-stream toe C being zero. The area of the triangle $C\ T\ S$ represents the total pressure on the base of the wall, the line $C\ S$ representing a uniform decrease of pressure on the base from T to C.

According to the deductions from the experiments cited above, the pressure is mainly transferred from T to the interior of the base, where the resistance to compression is much greater. The pressures at different points may be represented by the ordinates of the curve $C\ V\ T$, the area between it and the line $C\ T$ being equal to the area of the triangle $C\ T\ S$. I am unable to give the exact form of the curve, but some of its characteristics, to accord with what has been stated, are obvious. The ordinates at C and T must be small; the maximum ordinate must be less than $T\ S$, and be either within or near the middle third of the base. The curve $C\ V\ T$ has these characteristics, and is composed of quadrants $C\ V$ and $V\ T$ of two ellipses, the common ordinate $H\ V$ at the intersection of the resultant $g\ F$ with the base $C\ T$ being 0.6366, $T\ S$ being unity.

The sum of the areas of the two quadrants of ellipses is equal to the area of the triangle $C\ T\ S$, and consequently, represents the total pressure on the base of the wall.

By Table No. 3, in a wall 400 feet high, according to Debauve's method, the maximum pressure is at the down-stream toe T, where it is 390.64 pounds per square inch. If the pressure is distributed according to the curve $C\ V\ T$, the maximum pressure is at V, 88.88 feet from the down-stream toe, where it is $390.64 \times 0.6366 = 248.68$ pounds per square inch.

It will be understood that the curve $C\ V\ T$ is given only as an illustration of the distribution of pressures according to the theory advanced, and with no pretension to precision, either as to the distribution or the determination of the point where the pressure is a maximum. It would be desirable to be able to do so, but it is not essential to the present purpose; the main point in reference to the safety of the wall being that the vicinities of the toes are relieved from dangerous pressures by the transfer of a large part of the same to parts of the base capable of sustaining them without danger of crushing.

We have no means of estimating the amount of compression in different parts of the mass. It may be thought to be proportional to the pressure, but this is compensated for, more or less, by the increased strength to resist compression, depending on the increased lateral support to given areas as we recede from the exterior; the result tending to approach uniformity in the amount of compression at different parts of the mass.

In walls and dams to resist the pressure of water, the width of the base and the pressure on it are proportional to the height, and the part of the base capable of sustaining great pressure without crushing is substantially in the same proportion. If the theory of the transfer of pressure that I have attempted to establish is admitted, the conclusion that there is no danger of failure from crushing in any case must also be admitted.

In the case of the rock on which the wall is built, having seams containing water in communication with the water in the reservoir, by constructing drains sufficiently near to each other in the base of the dam to discharge the water from the seams through the outer toe of the wall, no upward pressure could be communicated to the wall; it would, however, allow some waste of water, and might be objected to as interfering with the homogeneity of the masonry. If drains are not provided, it may be questioned whether the upward pressure would extend beyond the parts of the masonry immediately in contact with the seams.

Assuming that the work is as well done as it can be, that is, that all the stones are perfectly bedded in the mortar, and that all the joints are solidly filled with mortar or concrete, would the upward pressure from the seams be communicated to the entire base of the wall, as is assumed, in the method of arriving at the required width of the base of the wall, by formula (12)? This involves the question as to whether the pressure of water can be communicated through the capillary channels in the mortar. Not finding anything definite on record on this point, I made the following experiments:

Fig. 8, Plate XX, is a section of the apparatus. The cast-iron case, *A A*, is an ordinary reducer for the purpose of connecting an 8-inch pipe with a 6-inch pipe. To fit it for this purpose, heads, *B B'*, were applied, packed with rubber, and kept in place by four bolts, two of which are shown at *C C*. An inner pipe *D*, 3 inches interior di-

ameter, is bolted to the head B', the joint being packed with rubber. The interior of the case A and the pipe D are filled, as represented, with Portland cement mortar, in the proportion of cement one, sharp pit sand two, the brand being, The Brick, Lime and Cement Company, Limited, Durham Works; trade-mark, Murston Works. The mortar was tempered to the usual consistency for use, and was carefully worked into place by stirring it with a rod. Spaces $E\,E$ and F, to receive the water passing through the mortar, were left at the bottom, which were filled with gravel, and covered with cotton cloth, to prevent the mortar from filling the spaces in the gravel. A space, G, was left between the top of the mortar and the upper head B. This space was connected with the system of pipes laid for supplying water for extinguishing fires in the Lowell factories, by the pipe K, and the spaces $E\,E$ and F were furnished with discharge pipes I and H, the outlets of which were furnished with stop-cocks or caps. Each of the pipes $K\,H$ and I was furnished with a pressure gauge L M and N, as represented in the figure.

The interior of the case A was originally protected from oxidation by Dr. Smith's patent varnish. It was feared that this might interfere with the experiment by preventing a perfect contact of the mortar with the inside of the case, and allow the water, under pressure, to follow the joint. To avoid this, the varnish was burnt off.

It is well known that Portland cement mortar expands in setting under water, which would seem to be a sufficient guarantee that the water would not follow the joint; but, as further security, the inner pipe D was put in, and means provided for testing the percolation of water through it, and the amount of pressure transmitted, independently of that transmitted through the mortar between it and the case.

The experiments consisted in noting the pressures at the several gauges at the beginning and end of periods, which gave sufficient time for them to become established. The pressure at the gauge L depended entirely on the pressure in the supply pipe, and was subject to changes of short duration, depending mainly on the drawing of water in a neighboring office. These changes were, however, so infrequent and of such short duration, that they had little influence on the results, and no account has been taken of them. The normal pressure at gauge L was about 77 pounds, and it is to be so taken in all these experiments.

The cement mortar was mixed and put into the apparatus February 28th, 1888, and remained in the air in a cellar, where the average temperature was about 36 degrees Fahr., for seven days. It was then put into a tub of water, in the same cellar, where it remained twenty-one days, the temperature of the water averaging 38.3 degrees. The apparatus was then taken out of the tub, and remained in the air another seven days, the temperature of the air being about 40 degrees. It was then returned to the tub, where it remained another seven days, the temperature of the water being 40 degrees. It was then taken out, and the mortar having become well hardened, the apparatus was completed for the experiments.

Unfortunately, the case *A* was not a sound casting, and there was a minute leak on one side, near *O*, and there were indications of other minute leaks through the head B^1, and at the packing between it and the case *A*. These leakages, undoubtedly, modified materially the results of the experiments on the transmission of pressure.

The packing of the inside pipe *D*, to the lower head *B'*, was tested under the full pressure before commencing the experiments, and was found to be perfectly tight.

The experiments were all made in the cellar, between April 11th and 25th, 1888; the average temperature of the air being about 40.5 degrees.

The pressure gauges were compared, and corrected to gauge *L* as the standard.

Experiments on the Transmission of Pressure through Hardened Cement Mortar.

Experiment 1. From April 11th, 6 o'clock P.M., to April 12th, 9.15 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 65.5 pounds. During the final one hour and twenty minutes there was no change in the pressure at gauge *M*.

Experiment 2. From April 12th, 9.16 A.M., to April 12th, 3.27 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 64.0 pounds. During the final one hour and thirty minutes there was no change in the pressure at gauge *M*.

Experiment 3. From April 12th, 3.28 P.M., to April 13th, 11.16 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from

0 to 62.6 pounds. During the final one hour there was no change of pressure at gauge *M*.

Experiment 4. From April 13th, 11.22 A.M. to April 13th, 5.35 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 59.75 pounds. During the final one hour and twenty-six minutes there was no change of pressure at gauge *M*.

Experiment 5. From April 13th, 6.2 P.M., to April 14th, 8.1 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 57.25 pounds. Period during which pressure at guage *M* was stationary not observed.

Experiment 6. April 14th, 11.42 A.M., to April 16th, 9.14 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 46.5 pounds. During the final one hour and nine minutes there was no change of pressure at gauge *M*.

Experiment 7. From April 16th, 9.30, to April 19th, 8.12 A.M. Pipes *I* and *H* both closed. Pressure at gauge *M* went up from 46.5 pounds to 72.5 pounds. Gauge *N* not then attached. During the final twenty-one hours and forty-five minutes no change of pressure at gauge *M*.

Experiment 8. From April 19th, 8.12 A.M., to April 20th. 10.45 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went down from 72.5 pounds to 39.75 pounds. During the final two hours and forty-two minutes no change of pressure at gauge *M*.

Experiment 9. From April 20th, 10.45 A.M., to April 23d, 11.30 A.M. Pipes *I* and *H* both closed. Pressure at gauge *M* went up from 39.75 pounds to 73.5 pounds. Pressure at gauge *N* went up from 0 to 71.0 pounds. During the final three hours and eighteen minutes no change of pressure at either gauge.

Experiment 10. From April 23d, 11.30 A.M., to April 24th, 3.40 P.M. Pipe *H* open, pipe *I* closed. Pressure at gauge *N* went down from 71.0 pounds to 58.0 pounds. During the final seven hours and thirty minutes no change of pressure at gauge *N*.

EXPERIMENTS ON THE PERCOLATION OF WATER THROUGH THE CEMENT MORTAR IN THE CASE *A*, FIG. 8, PLATE XX. CEMENT 1, SHARP PIT SAND 2, BLOCK 15.84 INCHES THICK, SIX TO EIGHT WEEKS OLD.

Experiment 1. From April 12th, 10.55 A.M. to April 12th, 1.58 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *L*, 77.0 pounds. Pressure at gauge *M* went up from 27.0 pounds to 64.0 pounds. Drip from pipe *I* in 183 minutes, 1.453 pounds. Rate of percolation in twenty-four hours,

$$\frac{1440 \times 1.453}{183} = 11.433 \text{ pounds.}$$

Experiment 2. From April 24th, 3.53 P.M., to April 25th, 3.53 P.M. Pipes *I* and *H* both open. Pressure at gauge *L*, 77.0 pounds. Drip from pipe *I* in the twenty-four hours, 8.256 pounds. Drip from pipe *H* in the twenty-four hours, 1.762 pounds. Total amount of percolation in the twenty-four hours, 10.018 pounds.

In the experiments on the transmission of pressure through the mortar, Nos. 1, 2, 3, 4, 5 and 6, there is a gradual diminution in the final pressure at gauge *M*, which indicates the pressure transmitted through the mortar between the water spaces *G* and *F*. During all these experiments the pipe *I* was open, and there was no pressure in the space *E*, which afforded a free outlet for the water percolating through the mortar between the pipe *D* and the case *A*; while in the space *F* there was a considerable pressure. The percolation through the pipe *D* must have ceased as soon as the space *F* was filled with water, if there was no leakage from it, which we have assumed to be the case.

The pressures in the mortar at the top of the pipe at *P*, and in the space *F*, must then have been the same, and equal to the obstruction to the percolation through the mortar between *P* and the space *E*. The course of the flow through the mortar, immediately over the pipe *D*, would be deflected to the space between it and the case, as shown by the arrows. The pressures indicated by the gauge *M* are, therefore, on the assumption that there was no leakage from *F*, the pressures in the mortar at the top of pipe *D*.

Our water supply contains more or less sediment, particularly at the season when these experiments were made; and to avoid interference

with the percolation through the mortar, from its deposition, the water was first passed through a sand filter. On examination of the surface of the mortar after the experiments were made, it was found to be covered with a fine red sediment, evidently, in part, rust from the pipes. This undoubtedly caused the reduction in the rate of percolation from April 12th to 24th, as shown in experiments 1 and 2. This reduction in the percolation, combined with the leakage, accounts for the diminution of pressures in experiments 1 to 6 on the transmission of pressure.

In experiment 9 the only outlet for the percolation was the leakage, and the pressure attained in the space F was within 3.5 pounds of the pressure in the space G, and the pressure in E within 6.0 pounds; and it is reasonable to assume that if there had been no leakage, the pressures would have been the same at all the gauges, except that due to their different elevations, which would give about 1 pound more at gauges M and N than at gauge L.

These experiments are on a small scale with an imperfect apparatus; but I think we are justified in inferring from them that pressure can be transmitted through hardened mortar composed of one part of Portland cement and two parts of sharp pit sand; and that in the base of a wall founded on a rock, having numerous seams, containing water under a head, the flow from which is not carried off by drains, an upward pressure may be transmitted through the mortar to the entire base.

The experiments on percolation indicate that water under pressure penetrates the mortar, and unless the up-stream face of the wall is protected by a waterproof lining, the mass of the masonry would be saturated. What effect saturation would have on the stability of the wall is a question, but it would appear to be an additional reason for draining the base, as it would be difficult to prevent the penetration of water at every point.

The formulas (7) and (12) and the corresponding Figs. 5 and 6, Plate XIX, are intended to be used only as skeletons or diagrams to be used in designing high walls or dams to resist the pressure of water, in which it is desired that the amount of masonry should be a minimum. The triangle $A\ C\ T$ being first drawn, the apex being at the highest flood line, additions can be made as desired to provide for all the contingencies to which the wall may be subjected, and the wants to be provided for; such as carrying the wall above the flood line, and adding to its thickness at the upper part to resist the pressure of ice and the action

of waves. Such additions will all add to the weight of the wall and increase its stability over that given by the diagram, which should be treated as an inscribed figure indicating the smallest admissible amount of masonry.

In dams across rivers, over which water is intended to flow, sometimes in great volume and carrying large masses of ice, a wide departure from the diagram will be required. It may, however, be of use in such cases as indicating the dimensions required to resist the pressure of the water.

It is often the case that in order to reach a suitable rock foundation a considerable part of the height of the wall is below the permanent level of the water in the ground. In this part of the wall the effective weight per cubic foot of the masonry is only its excess above the weight of a cubic foot of water. The diagrams computed as above consequently apply only to the part of the wall above the permanent level of the ground water; the part below may be considered as a foundation, to which a different computation applies.

Fig. 1.

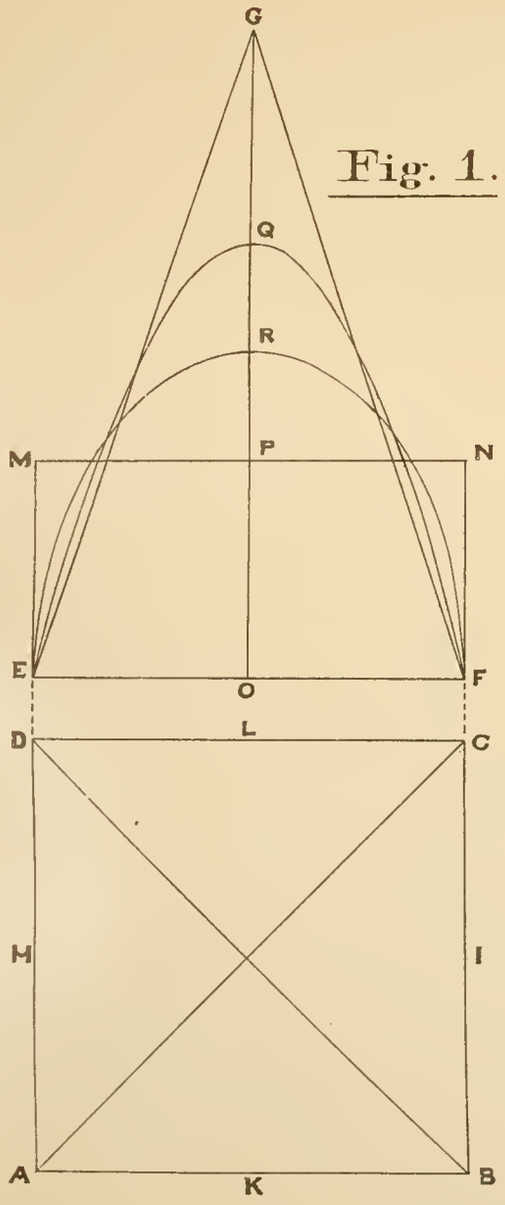

Fig. 2.

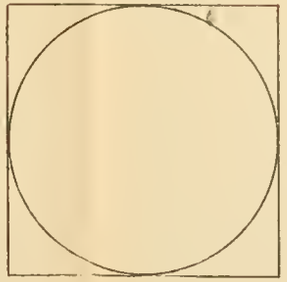

Fig. 3.

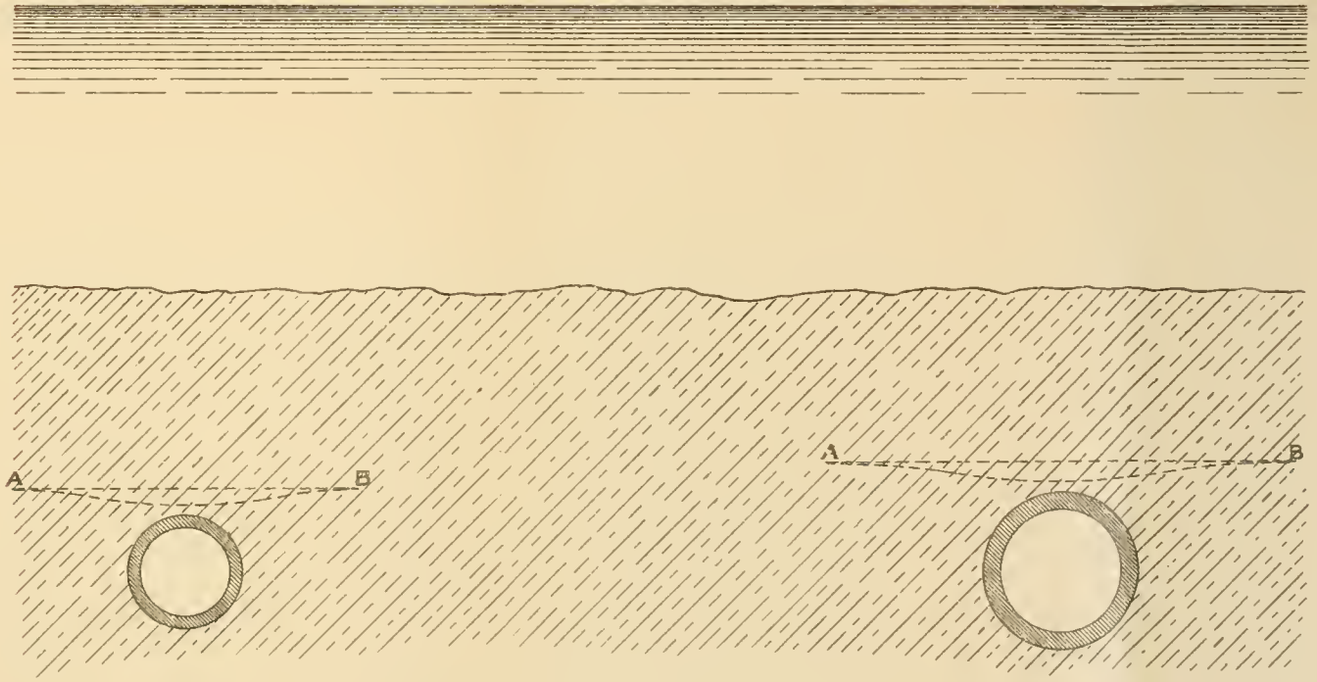

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Fig. 4

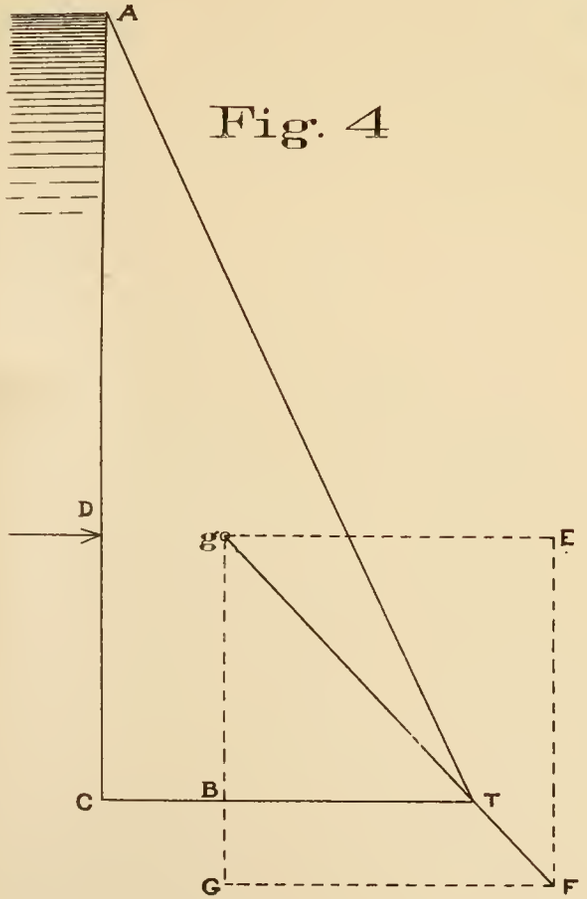

Fig. 5.

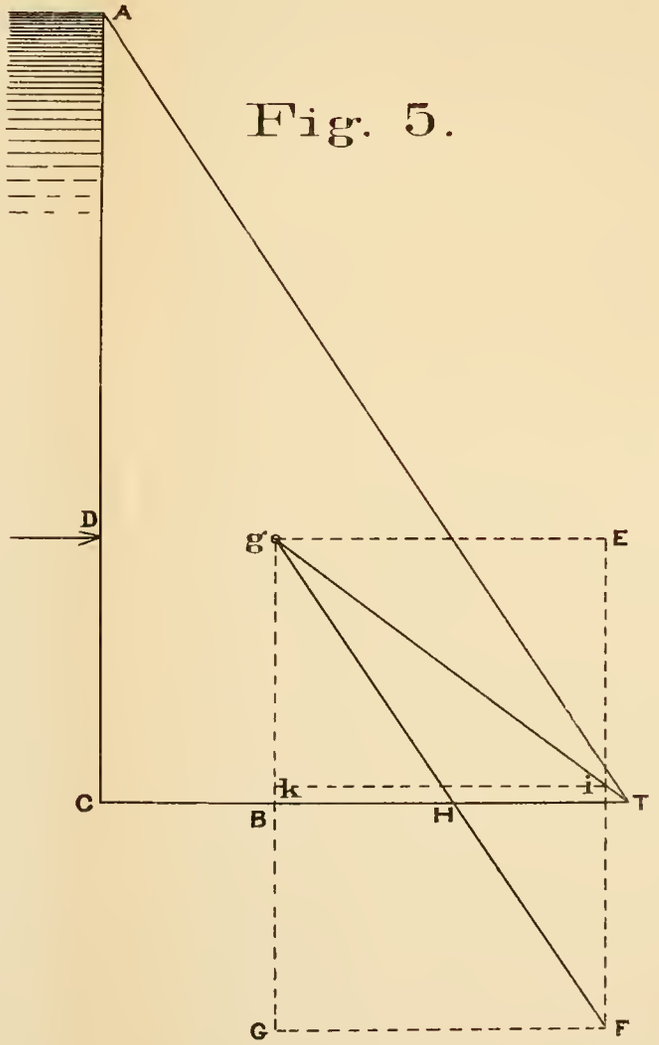

Fig. 6.

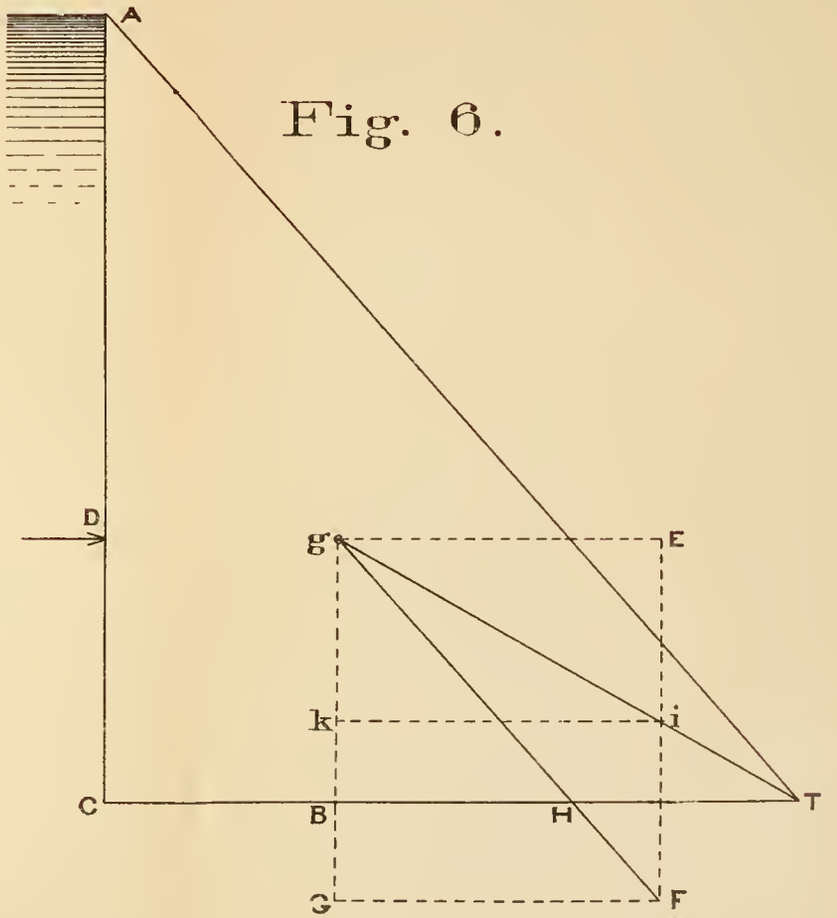

PLATE XX
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FRANCIS ON
HIGH WALLS OR DAMS

Fig. 7.

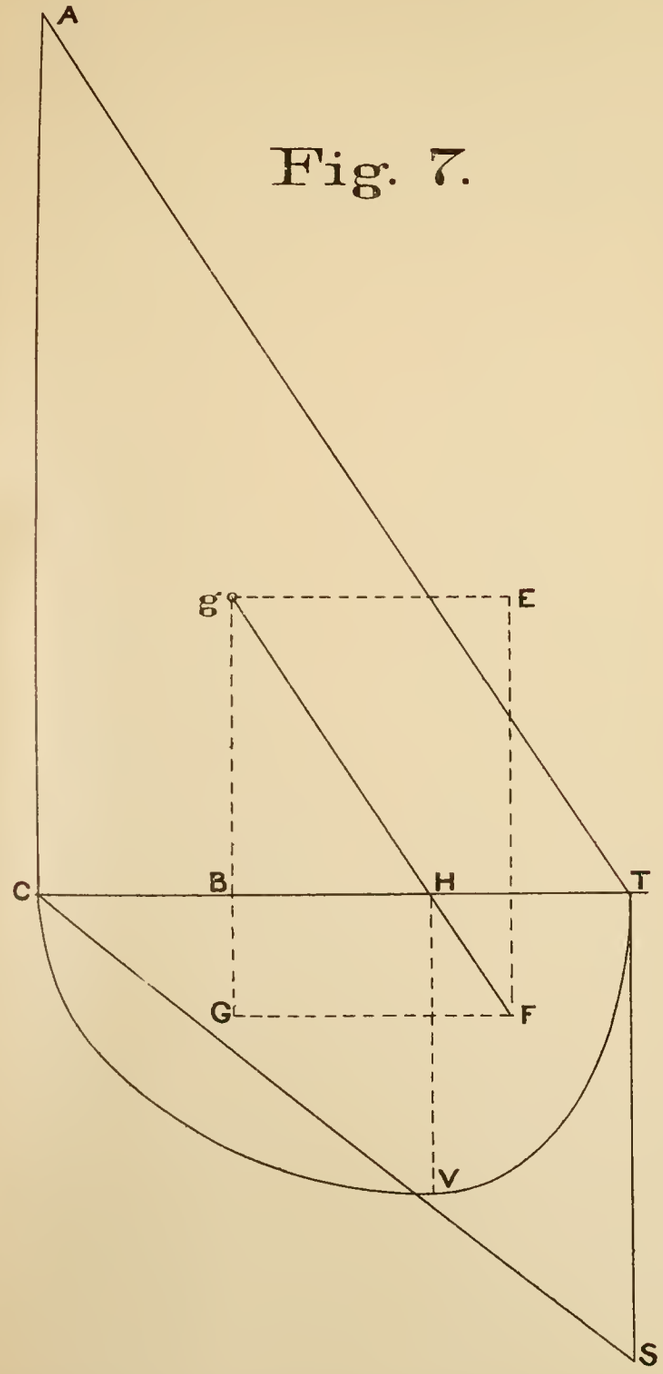
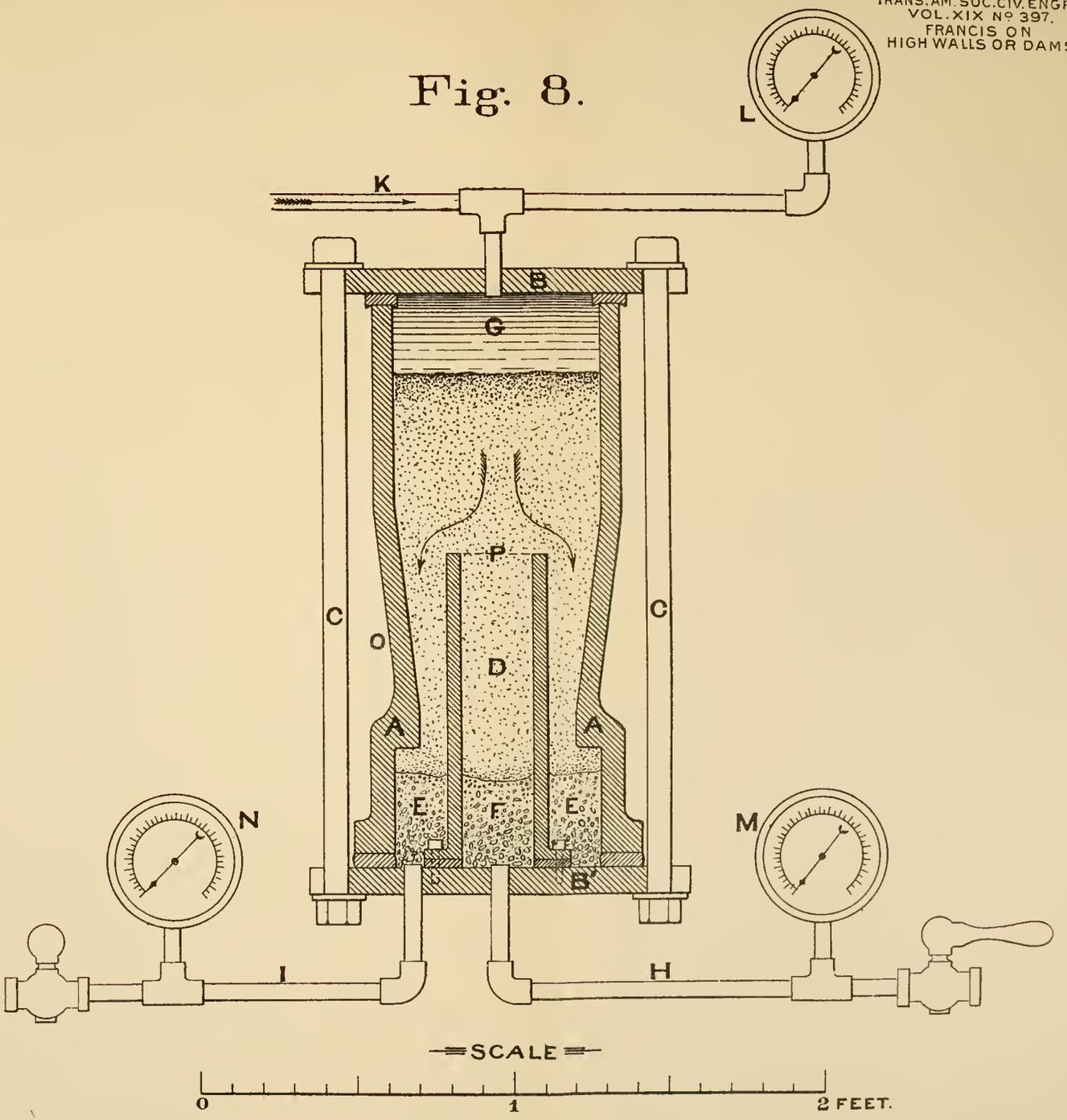


Fig. 8.



DISCUSSION.

EDWARD WEGMANN, Jr., M. Am. Soc. C. E.—The paper of Mr. Francis on high masonry walls or dams, involves the consideration of the following important topics:

First.—The distribution of pressure in a wall of masonry.

Second.—The proper form of profile for a masonry dam.

Third.—The effect of the perviousness of the masonry upon the stability of a dam.

With reference to the first of the above topics, Mr. Francis makes a wide departure from the views hitherto entertained. The usual formulas for the distribution of pressure in a wall have been derived by supposing the masonry to be rigid. Under this assumption, the pressures on the foundation must either be uniformly distributed or form a uniformly varying stress, giving the maximum pressures near the front or back face of the wall. While this hypothesis, which neglects the elasticity of the masonry, is evidently inaccurate, it leads to formulas which are certainly safe, as they exaggerate the pressures in the weakest parts of the wall, namely, near its faces.

In marked opposition to the assumption just stated, Mr. Francis considers that the pressures will be practically zero at the faces, and that they will distribute themselves in accordance with the power of resistance of the various parts of the wall. As a result of this view, he reaches, on page 156 of his paper, the conclusion that it is "unnecessary to take into account the crushing effect of the pressures, which we must do if the masonry is assumed to be perfectly rigid."

This new view of the distribution of pressure is based upon the following facts:

First.—The experiments of General Gillmore have shown that the resistance to crushing of hand specimens increases rapidly with the area of the bed subjected to the pressure—a result which is doubtless due to the additional strength the interior parts of the specimens receive from the lateral support of the outer portions.

Second.—The rock formations of the crust of the earth sustain safely pressures which would crush small specimens.

Third.—When a tunnel is driven through rock or even through the clay bottom of a lake, the pressures of the mass directly over the tunnel are safely transmitted to either side.

With reference to the experiments upon the crushing strength of hand specimens, I must state that they do not appear to me to present a very close analogy to the conditions of pressure in a masonry wall or dam. As these specimens were subjected to a uniformly distributed pressure between two practically rigid plates, it is evident that the outer weaker parts could not be crushed until the inner portions had been

considerably compressed. The experiments would have presented a greater similarity to the ordinary conditions in a course of masonry in a wall or dam if the specimens could have been crushed between two flexible plates, with the pressure varying from a given edge to the opposite one. Under such circumstances, it cannot be doubted that an outer portion of a specimen might have been crushed before the inner parts had been much strained.

To show that such a result might readily occur in a dam, we will consider the triangular reservoir wall, *A C T*, given in Fig. 7 of Mr. Francis' paper. The effect of the water-pressure against this wall is two-fold. It tends :

First.—To revolve it about the front (down-stream) toe *T*.

Second.—To slide it on the foundation or to shear it apart.

The first tendency evidently causes the pressures on the foundation to increase from the up-stream to the down-stream face of the dam. If the water-pressure could be increased until it finally turned the wall over, the whole pressure would be thrown on the front toe *T*, which would, of course, be crushed.

If we wish to represent the distribution of the pressure on the base graphically, the diagram of stress must fulfill the following two conditions :

First.—Its area must represent the total pressure.

Second.—Its center of gravity must lie in the line of action of the resultant pressure, that is, in the vertical line *H V*.

The triangle *C T S*, which represents the distribution of pressure for a rigid dam, satisfies both of the above conditions, and shows what might have been imagined, an increase of pressure towards the front face. The elasticity of the masonry will, undoubtedly, modify this diagram somewhat, curving the line *C S*, but it can never be like the diagram *C V T* given in Fig. 7 of the paper under discussion, as its center of gravity will evidently not be in the line *H V*, and will therefore not satisfy the second given condition. To obtain a rational pressure-diagram, which shall fulfill both of the requirements stated above, we must adhere very closely to the triangle *C T S*. As the moment of the pressures on the front third of the base *H T*, must balance the moments of the pressures on the other two-thirds, in order to keep the center of gravity of the stress diagram in the vertical line *H V*, the pressures on the base must, evidently, increase considerably from the back towards the front of the wall. Just where the maximum intensity of pressure will occur is unknown, but it must be near the front face.

If we now examine the case when the reservoir is empty, we shall arrive at similar pressure-diagrams, only that their positions will be reversed from what they were when the reservoir was full, the maxima pressures being now near the back face. Another way of arriving at the same conclusion is first to imagine the triangle *A C T* to be composed

of columns of masonry placed side by side. As the pressure on any part of the base would depend simply upon the weight of the column resting on it, the pressures would diminish from the back face to the front. Now conceive the columns of masonry to be united. The elasticity of masonry would undoubtedly modify the pressure diagram somewhat, but I do not think very materially, as the center of gravity of the diagram would have to be in the line *B G*.

The facts stated incline me to urge the adherence to the usual formula for the distribution of pressure. A wall having a symmetrical profile may possibly give rise to pressures like those shown in the diagrams for Fig. 1, but as our present knowledge of this subject is very meagre, I think the safer course is to exaggerate the pressures near the faces of a wall rather than to go to the other extreme by assuming them equal to zero.

The manner in which the pressures distribute themselves in a large mass of clay or rock, through which a tunnel is bored, does not seem to me to bear upon the case of a dam subjected to water pressure; for in the former case we have to deal with symmetrical forces which are, in the main, vertical, while in the latter we have to consider inclined forces, which generally necessitate an unequal distribution of pressure. The equilibrium existing between the thrust of the water, the weight of the dam and the reaction of the foundation is the important principle which determines this distribution, and I think the elasticity of the masonry can modify this only to a limited extent. The relative strength of the different parts of the wall cannot exert an important influence in this matter. I can, therefore, readily conceive how a given course of masonry which lies between two other elastic courses, may be crushed near one face before the strength of the inner parts is fairly exerted. I may also state here that when a tunnel is not subjected to symmetrical forces, or is weaker on one side than on the other, there will always be a strong likelihood of its caving in.

Let us now consider the second topic touched upon in Mr. Francis' paper, viz.: The correct profile for a masonry dam. The early writers upon this subject made their designs conform to the following two conditions:

First.—The pressures in the masonry or on the foundation must not exceed certain safe limits, whether the reservoir be full or empty.

Second.—The dam must have, at all depths, ample resistance to sliding or shearing.

In order to obtain the greatest economy, these writers designed what they called "profiles of equal resistance," that is, profiles in which the maxima pressures reached near either face remained uniform at all depths. So long as the assumed limits of pressure are moderate, say 6 to 8 tons per square foot, no objection can be raised to this method, but when we permit higher pressures—and the tendency has steadily been in this direction—some other condition must be introduced. We could

only design a profile having a uniform maximum pressure of about 14 tons per square foot at all depths, by allowing the lines of pressure in the upper part of the profile to be dangerously eccentric.

Professor Rankine has pointed out that in addition to the two conditions mentioned above, the profile of a dam ought to be designed so as to keep the lines of pressure, reservoir full or empty, practically within its center third. By complying with this requirement, we will assure at all depths a factor of safety of at least two against overturning and also assure ample strength against shearing. This condition of Professor Rankine is really the true principle upon which the design for a masonry dam should be based, the pressures near the faces being permitted to increase gradually from the top until they reach, at a depth of about 200 feet, a maximum of about 14 tons per square foot, a stress which has been safely sustained by the Almanza Dam for more than three centuries.

As no masonry dam 200 feet high has thus far been constructed, I must agree with Mr. Francis that, up to this limit of height, the crushing strength of the masonry need not be considered. But when we go beyond this and are to design a structure like the proposed Quaker Dam, then I think the limiting of the pressures, found by the usual formula, becomes the all important condition. To build a dam, 300 to 500 feet high, according to the triangular profile given in Fig. 7, Plate XX, and in Table No. 3, page 162, I think would be extremely hazardous.

It has been shown by Professor Rankine, Professor Intze and others, that so long as we have only to limit the lines of pressure to the center-third of a profile, the most economical form to withstand simply the hydrostatic pressure of the water, is given by a right-angled triangle with its vertical side towards the reservoir. As a dam must, however, have sufficient strength to resist the action of waves or shocks from floating bodies, the top of such a triangular profile must be reinforced by giving it a certain width and also a super-elevation above the highest water surface, which will depend upon local circumstances. Professor Castigliano has proposed, in the Politecnico for 1884, to convert a triangular profile, such as is given in Fig. 7 of Mr. Francis' paper, into a practical design, simply by adding the necessary top-width. While such a profile will have ample strength at all depths, it does not give the best distribution of the masonry in the wall, and, consequently, not the greatest economy.

A high dam ought to have a top-width of 15 to 20 feet. The proper manner of designing the profile is to take the given top-width as one of the data which must enter into the calculation.

If we follow this plan the theoretical triangular profile will be considerably distorted, so to speak, in the upper parts of the dam, but in the lower portions the faces will become parallel to those of the triangular form. The sections shown in Figs. 4, 5, 6 and 7, of the paper

under discussion must be regarded, therefore, as theoretical profiles, which have no great practical value.

I come now to the last topic contained in Mr. Francis' paper: The effect of the perviousness of masonry in a dam.

The experiments described are very interesting, and show certainly that the hydrostatic pressure of water can be transmitted through a certain thickness of good hydraulic mortar.

What effect will the water penetrating the wall exert upon its stability? So long as it passes through the wall and not under it, evidently none (leaving the deterioration of the masonry out of the question), for the water penetrating through any small channel will exert practically the same pressure downwards as upwards. This conclusion is verified by the experience with the Habra Dam in Algiers. When the water was first let into the reservoir the dam is said to have presented the appearance of a gigantic filter, so great was the leakage through it, and yet this fact does not seem to have affected its stability. This loss of water diminished rapidly owing to the channels of leakage becoming clogged.

After having stood for eight years, the dam was ruptured, but that was after an unusually severe storm, when the water passed about one meter high over the parapet of the wall.

Should the water penetrate between the foundation and the base of a dam, it would undoubtedly exert an upward pressure and diminish the stability of the wall.

The experience with the high masonry dams built abroad has shown, however, that with proper care and precautions, such walls, formed of ordinary rubble masonry, can be built so water-tight that only a slight dampness is noticed on the down-stream face. Taking this well-established fact in connection with the experiments of Mr. Francis, I should conclude that the water can penetrate but a short distance with its full hydrostatic pressure into the masonry, and that if it passes through a well-built dam, it loses its force, and simply goes through in a capillary manner.

It has been found that whatever leakage through the wall takes place when the water is first turned into a reservoir, diminishes rapidly, this applying, of course, also to the loss of water at the foundation.

I know of but one modern dam which has been designed on the supposition that the water pressure from the reservoir might extend under the whole base, viz.: the Gileppe Dam, built in Belgium in 1870 to 1875. This dam contains an excess of 75 per cent. of masonry over what is placed in modern French dams of the same height. I think that I am safe in stating that if the hydrostatic pressure due to the water in the reservoir could extend under the entire base of any of the recently built dams, the line of pressure would become so eccentric that the wall

would fail. In a properly constructed dam, I think, therefore, that any water leaking at the foundation is likely to have very little pressure.

If such be the case, and I think the experience with modern dams fully corroborates this view, then I should think, instead of providing drains in the foundation of a dam, as was done in the Vyrnwy Dam, or placing pipes in the masonry, as was done in the Tytam Dam, the idea being to get rid of any water that might filter into the structure, that such channels which tend to facilitate the infiltration ought to be avoided, and that efforts should be made to make the wall and the foundations as water-tight as possible.

To obtain this end, the masonry near the reservoir ought to be especially well laid, and it may also be advisable for very high dams to build a puddle bank of suitable material against the back face, for a certain height.

A. Fteley, M. Am. Soc. C. E.—The statements expressed in Mr. Francis' paper and by Mr. Wegmann in regard to the distribution of weight on the base of high masonry dams, are founded on the assumption that the whole mass of masonry is of a homogeneous character. In fact it cannot be so. Parts of the structure must necessarily be built at different times of the year, within wide limits of temperatures; various kinds of stones must be used, which will admit of varying quantities of mortars; mortars may be made of different ingredients; differences in the sizes of the stones and in the workmanship in various parts of the structure may occur. All these and other causes will modify the local power of resistance of the masonry and the mode of transmission of strains throughout the mass.

The opinions just expressed in regard to the distribution of weight on the base of the structure are widely different. The reference made by Mr. Francis to the action of lateral support which takes place in deep natural formations, is extremely interesting, and illustrates forcibly the fact that the limits of strains recommended for walls of moderate thickness can be safely extended for great masses of masonry.

Although unable to understand why the pressures exerted on the base of high masonry dams are minima at its extremities, I find it equally difficult to admit that the maximum strains are exerted at the end of the down-stream toe. The profile of many dams shows that the masonry generally ends on the down-stream side, in a point forming an acute angle with the horizon, and too weak to support the great strains indicated by calculation; if these strains had ever obtained at that point, a vertical fracture would have occurred at a distance from the toe where the masonry has sufficient section to resist the strain.

The absence of such fracture seems to indicate the absence of excessive strains at the toe.

Between these two extreme assumptions in regard to the distribution of the weight on the base of high masonry dams, there must be a middle ground which remains imperfectly defined.

In connection with the interesting experiment described by Mr. Francis, attention may be called to the fact that the whole mass of the dam is substantially as porous as the joint which unites it to the bottom, and that the weight of the water held in the masonry must influence the results of the calculations of stability.

It may be added here that all the computations are made without reference to the adhesive power of the mortar, which adds considerably to the stability of the dam.

Mr. Wegmann.—Mr. Fteley is undoubtedly correct in stating that the real distribution of pressure lies somewhere between the extreme views expressed by Mr. Francis and by me. As he cannot tell us, however, where that "middle ground" lies, I think we are compelled, in our present state of knowledge, to design the profiles of dams either by neglecting the pressures in the masonry entirely, as advocated by Mr. Francis, or else by calculating them in the usual way, assuming the masonry to be rigid and homogeneous. The hypothesis of rigid masonry leads undoubtedly to calculated pressures, which are in excess of those which actually exist in the wall. As regards homogeneous masonry, it is an ideal to be aimed at, although it may be never perfectly attained. I fail to see how we could obtain a satisfactory theory of dams based upon the supposition of heterogeneous masonry. Thus far all writers on the subject of masonry dams have based their formula upon the hypothesis of rigid and homogeneous masonry. The fact that the many high reservoir walls, constructed within recent years in conformity with this theory, have stood successfully, is an ample guarantee of the safety of the usual method of designing such structures.

Mr. Fteley.—I fully agree with the last speaker as to the advisability of submitting the problem to the calculations recommended by good authorities. They establish the value of maximum strains which the structure will not probably be called upon to resist; and if the design is so proportioned as to resist those maximum strains, the assurance that the pressures will remain within safe limits is of great value.

Theodore Cooper, M. Am. Soc. C. E.—As a listener to the preceding discussion, I agree in principle with both Mr. Francis and Mr. Wegmann.

Mr. Francis claims that the capacity of the mass is greater to resist strains at the center than at the toe. I thoroughly agree with him.

If, however, the capacity of the mass to resist compressive strain is greatest at the center and least at the toe, the point of danger is at the

toe. Mr. Wegmann is therefore correct in considering the most important point to be the toe pressure.

If the section of the dam be considered as a bracket maintained in its position by the superincumbent weight of material and resisting the overturning water pressure, we must obtain for equilibrium from the theory of equal moments certain pressures upon each element of the base which, combined with the pressure of the superincumbent weight, give the total pressure for this case and the maximum pressure on the toe.

This bracket cannot overturn without crushing the toe or shearing some portion of it off. I do not mean to say that the toe is going to take all the strain; every point is resisting, but this is the point of danger.

James B. Francis, Past Prest. Am. Soc. C. E.—I would like to add that we have an illustration in hydraulics. It is well known that water confined will bear a pressure only limited by the strength of the vessel containing it. Put a thousand pounds per square inch upon water in the cylinder of a hydraulic press, and there is no sensible yielding unless it bursts; it depends upon the lateral confinement. I think the same principle exists in great masses of masonry; the lateral confinement increases the supporting power; it all depends upon the strength of the lateral support.

Charles E. Emery, M. Am. Soc. C. E.—In considering the strains to which the masonry at the bottom of a high dam of this kind is subject, it seems to me that we must consider the fact that the material as a whole is of a porous nature, particularly the mortar, so that the pressure of the water is communicated through the mass by a series of cells more or less connected together. If the mortar adheres at all, the tensile strength of the walls of the cells aids the superincumbent weight to resist the lifting effect of the hydraulic pressure, and the thickness of the walls of the cells reduces the area upon which such pressure acts. The latter effect must also take place to some extent when there is no adherence of the mortar, as the superincumbent weight would hold certain points of contact under sufficient compression to exclude pressure, even if those points are so small as to represent more nearly the ultimate particles, and the pressure has an opportunity to circulate between the same in small passages, like the connected cells above referred to.

Some discussion has arisen as to the resistance which would be afforded by extending the down stream face of the dam out at the bottom in the form of a toe, one gentleman hinting that rupture would take place before it offered any considerable resistance. A thin toe run out in this way would, however, deflect more easily than a thicker mass of masonry, and the moment resulting from the forces required to produce such deflection would aid in maintaining the stability of the dam. This, together with the whole

question herein discussed, in regard to the pressures upon the masonry at the front and rear of the dam, illustrates only the usual phenomena in relation to the flexure of a beam. If a post, with the upper part of the same section as that of the dam, were set rigidly in the rock, a side-pressure corresponding to that of the water, would simply bring tensile strains on the side where the pressure is applied and compressive strain on the other; and were the beam without weight, the integrated moments of strains on each side of the neutral axis would be equal and opposite. As soon as we consider the beam to have a weight equal to that of the dam, the algebraic difference of the two moments when water pressure is considered, must be the same as before. The pressure of water would simply increase the strains on one side of the neutral axis and decrease them on the other, but the arithmetical sum or algebraic difference of the integral of the moments from the neutral axis, of the increased strains on the down-stream side, and of the integral of the moments of the forces relieved on the up-stream side, would equal the moment of the overturning force produced by the pressure of water. Ordinarily the material in a dam would be so disposed that the compressive strains on the up-stream side would never be relieved entirely so as to produce tensile strains in the masonry.

H. W. Brinckerhoff, M. Am. Soc. C. E.—Should not the lowest part of this curve, *C V T* (Fig. 7, Plate XX), be to the right of the line *H V*, since the center of gravity of the area included by the curve should be upon the line *H V*. The center of pressure is upon that line, and it would seem that the center of gravity of any figure representing that pressure should also be upon that line.

Mr. Francis.—I have not been able to find out where the center of pressure is; I have the general idea that there is a transfer of pressure from the outside towards the inside, but exactly how it would be distributed I have not been able to determine, and we have no exact data. I suppose the maximum pressure would be somewhere near that line.

Mr. Brinckerhoff.—Not the maximum pressure, but the center of gravity of the figure. I feel convinced myself that the center of gravity of the figure must be on the line *H V*.

Mr. Francis.—My idea was that the center of pressure would be on that line, but I can give no proof of it.

Mr. Brinckerhoff.—No, sir; if that pressure is supported at *H*, it would be in equilibrium, and there would be no rotation; if it is supported by any number of forces whose center of gravity is not at *H*, there will not be an equilibrium.

Mr. Francis.—I have based my estimate upon the excessive pressure near the outside being transferred towards the center, which is more capable of supporting it.

Mr. Brinckerhoff.—To assume the center of gravity of the area *C V T*

to be on the line *H V* would strongly affect the shape of the enclosing curve, and, I think, materially increase the maximum pressure.

To resist the same force of pressure applied at the same place, not only must the area of *C V T* (Fig. 7) be equal to that of *C S T*, but its center of gravity must be on the same perpendicular line *H V*. In order to fulfill this latter condition, the maximum ordinate of *C V T* must be some distance to the right of *H V*, and to fulfill the former condition, the maximum ordinate of *C V T* must be considerably greater than *H V*; in fact, the figure *C V T* will really differ much less from *C S T* than has been assumed.

Mr. Francis.—The first condition of the curve is that its area must be equal to the area of the triangle. Exactly what the form of the curve would be, and where the maximum of pressure would be, I have not determined ; I hope somebody else will ; I have not been able to.

George S. Greene, M. Am. Soc. C. E.—In the Yosemite Valley there is a vertical face of wall of some 3 000 feet in height. Now, how does this masonry stand all this pressure ; how has it stood all that it has for ages? It is because the pressure is transmitted through the entire wall, and the base is supported against crushing by this pressure on the lower courses, resisting any lateral movement, which movement must take place if there is any crushing. And so it would be in any well-built dam. There will not be this destructive pressure upon the beds, and I don't see that we can get any pressure that will crush granite when confined laterally.

A Member.—We must consider the crushing strength of the mortar ; it is, of course, the weakest part of a masonry dam. It may be that a pressure of twenty tons per square foot, or perhaps one of thirty might be tried, but it would be a hazardous thing for a person to design a dam on this supposition.

Gen. Greene.—How is it we have natural cases where a mass of masonry does stand at this height?

A Member.—I should say that the crushing strength of the mortar would have to be considered.

Mr. Francis.—In hydraulic presses the only limit to the weight that can be put upon the water is the strength of the vessel containing it.

Gen. Greene.—Did Mr. Francis use thin mortar or cement? Was it filled in? Was it tamped with a ram?

Mr. Francis.—It was mixed with the consistency that mortar usually is ; it could not be rammed. The proportions were sand, two; cement, one. It was put in as well as I could put it in, working it with a stick.

Gen. Greene.—I think you have had some experience as to the imperviousness of cement?

Mr. Francis.—Yes ; I think you can make a wall more impervious by a plastering of pure cement on the face next to the water than by any thickness of masonry as ordinarily laid.

G. Bouscaren, M. Am. Soc. C. E.—In testing small blocks of stone to destruction, if the pressure is applied gradually, failure begins almost invariably by the splitting of the blocks in a direction parallel, or nearly so, to the direction of the pressure, beginning near the faces, and extending rapidly toward the center of the block, as shown by the successive lines of division, $a\,a'$, $b\,b'$, in Fig. 1, Plate XXI. An explanation of this may be found in the fact that in all stones the ultimate resistance to tension is much less than the resistance to compression. A pressure P being applied to a stone block will create an elastic tension T at right angles to the direction of the pressure; when the limit of resistance to this tension is reached, the block will part, as illustrated, before the ultimate resistance to crushing has been actually reached.

The forces acting on all points, o, in the block (Fig. 2) are a pressure p, its reaction p', and the elastic forces t t, equal and opposite in direction, and at right angles to p; the only external forces opposed to t are the friction of faces $A\,B$ and $C\,D$ on their bearings.

Let us assume the block $A\,B\,C\,D$ to be a cube; let

a = side of cube.
$d\,w$ = element of surface of section through $m\,n$.
x = distance of $m\,n$ to face $B\,D$.
t = elastic tension per square inch.
Δ = ultimate resistance per square inch to tension.
p = pressure per square inch applied.
f = co-efficient of friction of faces $A\,B$, $C\,D$ on their bearings.

The block will part on line $m\,n$ when the sum of the elastic tensions on section $m\,n$ equals the ultimate resistance of the same section to tension plus the friction of surfaces $m\,B$ and $n\,D$.

$$\int t\,d\,w = \Delta\,a^2 + 2a\,x\,p\,f$$
$$t\,a^2 = \Delta\,a^2 + 2a\,x\,p\,f$$
$$t = \Delta + \frac{2\,x\,p\,f}{a} \qquad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

We may assume $t = K\,p$, K being a constant, hence

$$K\,p - \frac{2\,x\,p\,f}{a} = \Delta$$

$$p = \frac{a\,\Delta}{a\,K - 2\,x\,f} \qquad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

For $x = o$, $\qquad p' = \frac{\Delta}{K}$.

This is the expression of the limiting pressure (minimum) per square inch at the edges of the block.

For $x = \frac{a}{2}$ $\qquad p'' = \frac{\Delta}{K - f}$.

This is the expression of the limiting pressure (maximum) on the center of the block. For $K = f$, p'' is infinite; this means simply that for $K = f$ the block could not split in the middle, but must fail at that point by direct crushing, unless it spalls off progressively from the faces, as it generally does in fact.

From this explanation it is easy to understand why, in testing blocks to destruction, the average ultimate pressure per square inch increases with the area.

I think it is a more rational interpretation of the facts than to assume with Mr. Francis that the co-efficient of elasticity of the material increases from the faces to the center of the block.

In a wall (Fig. 3, Plate XXI) subjected to an eccentric resultant force F, having a movement around the center line $G\ G$ of its base, the maximum pressure per square inch must necessarily occur on the line of the down-stream toe B, if the co-efficient of elasticity is supposed to be constant throughout the wall.

Let e be the eccentricity of the resultant force F, and F_V its vertical component; R = stress per square inch at any point o of the base at a distance x from the center line; R' and R'' the stresses at A and B.

$a = A\ B$ = width of base in inches, the length of same Bc being equal to one.

Ω = area of base $A\ B\ C\ D$.

I = moment of inertia of $A\ B\ C\ D$.

m = moment of resultant force F around center line $G\ G$.

If the material of the wall is supposed to be perfectly elastic, with a constant co-efficient of elasticity,

$$R = \frac{m\ x}{I} + \frac{F_V}{\Omega} \qquad \ldots\ldots(3)$$

Substituting for m and I their value,

$$m = F_V\ e$$

$$I = \tfrac{1}{12}\ \Omega\ a^2$$

We have $R = \dfrac{12\ F_V\ e\ x}{\Omega\ a^2} + \dfrac{F_V}{\Omega} = \dfrac{F_V}{\Omega}\left(\dfrac{12\ e\ x}{a^2} + 1\right)$ $\qquad \ldots\ldots(4)$

For point A, $x = -\frac{1}{2}\ a$; $R' = \dfrac{F_V}{\Omega}\left(-\dfrac{6\ e}{a} + 1\right)$ $\qquad \ldots\ldots(5)$

This is the minimum value of R.

For point B, $x = \frac{1}{2}\ a$; $R'' = \dfrac{F_V}{\Omega}\left(\dfrac{6\ e}{a} + 1\right)$ $\qquad \ldots\ldots(6)$

This is the maximum value of R.

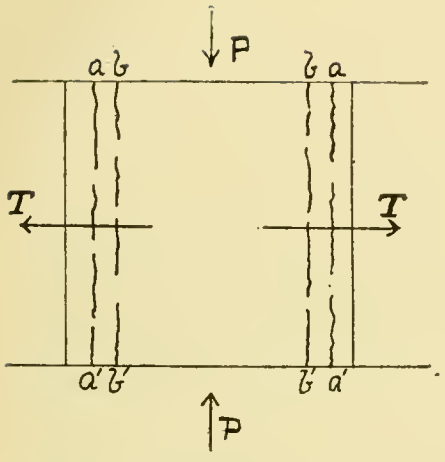


FIG. 1.

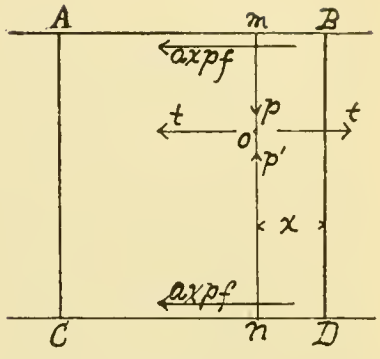


FIG. 2.

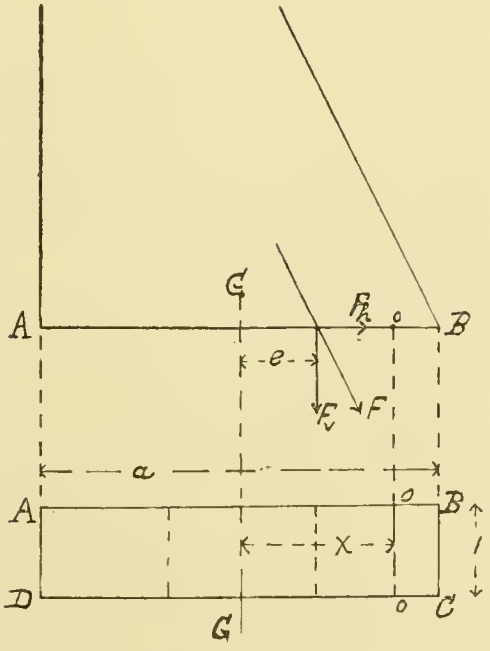


FIG. 3.

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PLATE XXI.

If we suppose the force F applied at a distance of $\frac{1}{3}$ of the width of base from the down-stream toe, $e = \frac{1}{6}\, a$; substituting in (5) and (6), we have

$$R' = o$$

$$R'' = 2\,\frac{F_V}{\Omega}$$

The pressure per square inch is zero on the up-stream toe of the dam, and double the averages on the down-stream toe.

For $e > \frac{1}{6}\, a$ we would have tension at A; if the resistance to tension cannot be depended on this must be avoided, as any crack in the up-stream face of the wall, by allowing the water pressure to exert itself in an upward direction on the base, would greatly diminish the stability of the wall, hence the necessity of confining the position of F to within the middle third of the width of base.

It is difficult to understand how, under such conditions, the intensity of pressure per square inch could ever be greater in the interior of the wall than on its down-stream face, unless by failure of the material forming the face.

As long as the elasticity of the material is preserved, the maximum stress must be met with at the face, where the material being free to spall off, as has been explained to be the case for small blocks, the resistance should be assumed to be the same for both cases.

This has been the practice generally followed by designers of modern masonry dams, and it does not appear that a departure therefrom can be justified on the theory advanced by Mr. Francis.

It seems that a better way of economizing material in high dams, would be to find a mortar of greater strength than the Portland, to be used for the masonry of the face.

Joseph P. Frizell, M. Am. Soc. C. E.—The experiments of Mr. J. B. Francis, on the percolation of water through cement mortar, though the results are what might have been confidently predicted, have an important bearing on the subject of high dams of masonry. Engineers of standing have affirmed that in computing the stability of cement masonry under water, when it is bedded on firm rock, the buoyancy of the mass should not be considered, the water being excluded from the bottom by the mortar. These experiments show that such exclusion is impossible.

Another important deduction might be made. Under a dam of this description a trench of greater or less depth is sometimes excavated and filled with concrete or other masonry, supposed to be more impermeable than the body of the rock. Such a feature placed near the up-stream toe of the dam is undoubtedly an element of security. It does not, however, follow that the introduction of a second trench, further down-stream, would be an additional element of security. On the contrary,

it would be a positive, and in certain supposable conditions, a very formidable element of insecurity. Consider the case in which all the conditions are unfavorable, viz., the section of the dam substantially a right-angled triangle; the bed rock seamy; one trench along the up-stream toe, the other along the down-stream toe. Add the supposition that the up-stream barrier is defective, the down-stream one tight, the whole bottom of the dam would be exposed to an upward pressure nearly equal to that due the head, and in the aggregate approaching the total weight of the dam. This would be a combination of circumstances not likely to occur in practice. Nevertheless, as suggested by Mr. Francis, no dam of this description, as ordinarily constructed, can be wholly free from action of this kind. The practical disposition suggested by these considerations, is to limit the strictly water-tight construction, both above and below the surface of the rock, to a few feet in thickness of the up-stream face, and make arrangements for free egress to all water that passes this barrier.

Such an arrangement would have advantages other than statical, which would become apparent in the course of time. The down-stream face of a high reservoir dam is fully exposed to the action of frost, which, as every one knows, acts much more rapidly in the disintegration of rock, when the latter is saturated with water, than when it is dry.

E. Sherman Gould, M. Am. Soc. C. E.—A very able and important paper. The observations upon the crushing of stone masonry are excellent. There can be no doubt—indeed the proof rests upon direct experiments—that resistance to crushing increases with the size of the block crushed.* It may be remarked in passing that this increase of resistance does not seem to hold good for tensile strains, for Mr. Clarke, in his experiments on the Boston Main Drainage Works, found no appreciable difference in the elementary resistance of cement briquettes of different breaking sections. Since the mortar is generally the weak part of masonry structures, it would appear that this should be tested, instead of the stone. Also, that the specimen cubes should be kept under sustained pressure for several consecutive days. No doubt the material would show very much lower results, both in tension and compression, under this treatment.

As Mr. Francis well points out, any force impeding lateral expansion retards rupture from pressure. A familiar application of this principle is the ringing of pile heads to prevent splitting under the blows of the driver.

Several practical consequences result from these facts: It is evident, as Dupuit points out, that a large stone supporting the foot of an iron column of much smaller section, will sustain a much greater pressure per square inch than a smaller stone of equal section with the foot of

* This was written before seeing the Watertown experiments, which I do not touch upon here, as Mr. Francis refers to them at the close of the discussion.

the column and entirely covered by it. It is also, I think, reasonable to suppose that a very deep foundation wall, particularly if built with vertical faces, and closely imprisoned by compact earth, is less liable to crush under pressure than a similar wall standing in relief upon the surface of the ground. Also, the exposed bed-arrises of ashler face work will be more likely to check if dressed to a sharp right angle than if worked to a pitched draft with a rock-face projecting immediately beyond it, so as to form an obtuse angle, and thus buttress the arrise, as it were, by an enveloping mass.

The following remarks upon this subject, translated from the treatise on stone bridges, forming the 10th fascicule of Mons. Debauve's Manuel de l'Ingénieur, will be heard with interest:

"Crushing is due, as Dupuit remarks, to the lateral expansion of the stone, the compressed parts endeavoring to escape at the sides, where they find no impediment to so doing. When the pressure becomes so great as to destroy the cohesion of the fibres, they burst asunder, and crushing ensues.

"Any cause which opposes this lateral expansion will have for effect the retardation of the crushing of the material, by annulling, more or less, the tendency to spread. Mons. Pelletreau, in a recent paper, corroborates the above statement, by demonstrating the following proposition: Other things being equal, a prism of stone resists crushing in proportion to the degree of friction existing between its base and the surface on which it stands."

The case of the sustaining power of the top and sides of the excavation for the Chicago tunnel, could no doubt be fortified by many similar instances. It appears to me that in the case of the larger tunnel, we have an opening 9 feet wide, spanned by an arched girder of stiff clay with horizontal extrados, 28 feet thick at the centre, fixed at both ends, and sustaining a nearly uniformly distributed load of about 6 000 pounds per running foot, including its own weight.

The formulas given by Mr. Francis for the proper length of base of a theoretical triangular wall, are very good and useful, particularly No. 12, which gives this dimension under the most unfavorable conditions of pressure.

In speaking of the intensity of crushing strains, Mr. Francis refers to the well-known formulas for calculating the pressures upon the two toes of a reservoir, when the resultant does not cut the base in the middle. These formulas are given, with considerable development of the principles upon which they rest, by Mons. Debauve in the work named above, and are also demonstrated, he states, by other methods by Mons. Bresse in his *Cours de Mécanique Appliquée*. They are founded upon certain assumptions of homogeneity and rigidity, which we know cannot be fully justified by facts, but without which the problem could not be dealt with mathematically. Indeed it may be asked whether we

do not concede all that these assumptions demand when we admit—as I believe we always do—that the elementary, or unit strain is uniform throughout the base of a wall, symmetrical about a vertical axis.

Practically, I think we shall find that profiles of equal resistance, determined by these formulas, commend themselves to the eye as possessing strikingly appropriate proportions; and, when joined to a moderate limiting elementary pressure, have given rise to structures which have proved themselves eminently fitted for the purposes for which they were built.

The views of Mons. Debauve himself upon this subject are expressed as follows, in a letter which I received from him a few years ago: "These formulas should be employed with great discretion, since they assume that the masonry is homogeneous, and take no account of its resistance to traction. Their use should therefore be restricted to cases where the resultant of pressures does not move greatly from the middle of the base, and we should always endeavor to bring this resultant into the neighborhood of the middle of the sections, as is done in the profile of large masonry dams."

If I understand Mr. Francis aright, he advances the proposition that if a dam, of no matter how great a height, be properly proportioned to resist overturning, it cannot fail either by sliding or crushing. The French authors, on the other hand, as far as I am aware, test for sliding, but take no account of overturning, thus tacitly asserting that if the wall be safe as against crushing, it cannot overturn. Perhaps we have here only two different ways of saying the same thing, in which case Mr. Francis' way of putting it is certainly the simpler.

In connection with the question of upward pressure upon the base caused by water running through fissures in the rock, Mr. Francis suggests that drains might be constructed in the base of the dam to carry this water away. Mr. Francis intimates, however, that they might be objected to, upon certain grounds. I think they would be highly objectionable features in a structure, the very nature of which should be unbroken compactness. If sufficiently large and numerous to be of service in the way intended—and this would have to be guessed at in their construction—they could not fail, if the feed were considerable, to occasion a very large loss of water, as they would constitute just so many important leaks in the dam. Moreover, I do not see that such drains would prevent the communication of upward thrust (although they might diminish it by changing hydrostatic to hydraulic pressure), unless all the water liable to reach the base in this manner were caught in drain-pipes sufficiently strong of themselves to resist bursting pressure, which would be manifestly impracticable.

As regards the experiments upon percolation and pressure transmitted through mortar I cannot say that I consider them satisfactory. I do not think the apparatus used was one fitted to give trustworthy results, even if

the leakage mentioned did not exist. I do not think that the removing the varnish from the inside of the reducer, and inserting the central pipe, were sufficient guarantees that the water did not pass around, instead of through, the mortar, at least to a considerable extent.

I think it may be doubted if the concluding paragraph of Mr. Francis' paper, regarding the loss of weight of foundations carried below the permanent level of water, is to be taken as an unquestioned fact. It would of course be safest, in such cases, to make calculations on both assumptions, and take the result which indicated the heaviest work.

W. W. Maclay, M. Am. Soc. C. E.—Some experiments recently made, as a member of the Committee on Compressive Strength of Cements, may be of interest in relation to that part of the paper of Mr. Francis, giving his experiment and deductions on the transmission of pressure through hardened cement mortar.

A brick of Portland cement mortar made by weight, 1 part Portland cement, 2 parts standard sand (crushed quartz passed between sieves of 400 and 900 meshes per square inch), one year and two weeks old, was placed in Fairbank's machine, at No. 88 Thomas street, of this city, used for forcing water through building brick.

Under a pressure of 75 pounds per square inch, kept up for six hours, the brick was practically water-tight except that, towards the end of the six hours, the packing leaked a little, and when this was tightened the leak stopped.

The portion of the bottom surface of the brick not covered by the packing, and exposed to the water pressure, was a rectangle 1¾ x 5¾ inches.

A brick of Portland cement mortar, 1 part cement, 1 part sand, one year old, fifteen minutes under pressure of 75 pounds per square inch, showed no signs of leaking.

Bricks of Portland cement mortar, 1 part cement, and 3 and 4 parts sand, one year old, both leaked freely under hardly any pressure, the gauge indicating less than 1 pound per square inch.

F. Collingwood, M. Am. Soc. C. E.—Some of the experiments made at the Watertown Arsenal, bear out the statement made by Mr. Francis in the paper just read, viz., that the central portion of a block is what gives the greater part of the strength to resist compression. Taking the National Portland cubes, mixed 1 to 3, and tested at forty-six months' age, we find at 200 pounds pressure per square inch, 8-inch cubes compressing an average on 12 inches, length of 0."0048, 12" cubes ."0016; 16" cubes ."0011; at 700 pounds, ."0102, ."0052, and ."0040, etc.; the strengths at rupture being 2 478, 2 434 and 2 520 pounds, respectively. When mixed, 1 cement, 3 sand, 6 broken stone, at 200 pounds, the com-

pressions were for same sizes, respectively, .″0054, .″0028 and .″0015; at 700 pounds they were, .″0100, .″0072, .″0046, and the strengths at rupture, 3 027, 2 690 and 2 979 pounds.

The cubes of neat Portland give the same results as to strengths; twenty-two months cubes, 8, 9, 10, 11, 12 inches, give averages at rupture of respectively 4 781, 4 745, 4 761, 5 381 and 5 439 pounds per square inch, but the compressions do not run so regularly.

Brick piers laid in Portland cement mortar, mixed 1 to 2, do not follow the law. An 8-inch pier, 16 inches high, fifteen months old, ruptured at 3 776 pounds, and one twenty-three months old, 80 inches high, 2 249 pounds per inch square.

12-inch piers, twenty-four months old, 120 inches high, broke at 2 000 to 2 250 pounds; a 16-inch pier, twenty-four months old and 121 inches high, at 1 887 pounds. The compressions were approximately uniform for all, and as shown, the small piers were as strong as the large.

The experiments on these piers show that the compression does vary quite uniformly with the pressure, nearly up to the point of rupture. and there will be undoubtedly, as stated, a tendency for the pressure in a dam to approach uniformity in various parts of a given section. Before the pressure at the toe could reach double that at the up-stream face, the compression would cause a yielding, and the interior would have a greater portion of the pressure thrown upon it.

The experiments on transmission of pressures through the mass seem to be conclusive, and certainly point to the necessity of taking account of the lifting force of the water. Of course the amount of water penetrating into the masonry, and held there, at any one time, adds its weight to that of the masonry, and is, in so far, an element of safety; but this is small. Any means by which the head can be destroyed would remove the pressure; and this leads to interior drains, such as have been provided in the Vyrnwy Dam. But without careful work in construction, such drains might prove of themselves an element of danger.

While the portion of a dam below the level of ground-water may be treated as a foundation merely, it is undoubtedly a part of the dam, and it would seem should be so considered, although requiring a modification in whatever formula is employed, if all circumstances be taken account of.

The level of ground-water below the dam is likely to be a variable quantity, and in time to come might be decidedly changed.

The conclusion that might at first glance be drawn from this experiment would be that in every case the full pressure due to a column of water having a base nearly equal to that of the dam and a height equaling that in the dam, would be acting to overthrow the dam, in addition to the horizontal pressure of the water against it. This cannot be true

unless the lower face of the dam can be made absolutely impermeable to water. Under actual conditions (supposing a concrete dam) the water will penetrate horizontally throughout the mass, and the pressures at a vertical plane at any given distance from the inner face of the dam will all bear a constant ratio to those at the inner face; that is to say, a "free surface," or zero at the top, increasing to a maximum at the bottom, but with no unbalanced upward reaction, unless the strata underneath the dam be more permeable than the dam itself. The pressure at any point will be, instead of that due to the height (h), that due to $\frac{h}{r}$, and will diminish as the distance of the vertical plane from the inner face increases. It becomes zero at the plane where the water leaves the mass, in this case probably by evaporation.

Now, in just so far as the dam is less permeable at some points than others, or in so far as there are direct channels underneath, or a more permeable material underneath, will there be an unbalanced upward pressure, which must be considered.

We have, therefore, the two limits: either the full upward pressure acting, which would certainly be an extreme case, on the one side, or no unbalanced pressure, which is probably as extreme on the other.

E. A. Fuertes, M. Am. Soc. C. E.—I can discuss, in a very brief manner only, the paper of Mr. Francis on the form of "High Dams." As is to be expected from this distinguished engineer, his paper has an important and useful practical bearing. His views on this difficult subject are entirely correct in most respects, and probably in all respects, if the ambiguity resulting from some of his conceptions of the elastic limit of stone could be explained by defining the sense in which he deals with this property of matter. In order to avoid falling into error on account of an insufficiently precise definition of the "modulus of elasticity," as applied by Mr. Francis in this case, I have consulted with my colleague, Professor Church, whose familiarity with mechanical laws makes him one of the highest authorities on this question. Mr. Francis is correct, according to Professor Church, if he defines "modulus of elasticity" as "the quotient obtained by dividing the stress per unit area at any instant, by the contraction per unit length which the stress has produced, irrespectively of whether the elastic limit has been reached or not."

Under the above definition, and granting that the compressing plates do not change their form, and remain parallel as they compress the specimen, Mr. Francis' deductions are entirely accurate; and the "modulus of elasticity" of the stone increases from the edges towards the center. Therefore, in order to produce uniform contraction at all points, the central pressure must be greater at the center than at the edges; and, consequently, the average pressure per unit surface ob-

tained by testing specimens of various areas will be greater as the area subjected to compression increases. Mr. Francis is also correct in ascribing the result to the support which each elementary supporting column derives from its neighbor ; which support must become a maximum at the center of the specimen compressed. Without wishing to be hypercritical, I will say that the transverse section of dams employed by Mr. Francis, as representing the form most economical of masonry, is not strictly correct, though it is admirably suited for the points he wishes to make, when proper deviations from this form are made to suit the cases of variable pressures and changes due to time, accumulation of material heavier than water on the up-stream face, and the widely different conditions of the foundations of dams.

It is true that in reference to all the mechanical conditions of stability, similar relations exist at each horizontal stratum of any dam ; but in the triangular dam the area of these strata are proportional, arithmetically, to their distance from the top of the dam, while the resultant pressure diminishes as the square of this dimension ; and, therefore, the triangular section gives an excessive cross-section against crushing in the upper beds of the dam.

Mr. Wegmann's recent book on the Quaker Bridge Dam (which is as yet the best thing written on this subject in any language) develops an exact method for determining the profile of dams on the accepted trapezoidal formula, and in a manner entirely simple and practicable.

The following criticism is made by Professor Church : "The conception of a factor of safety, as inferred from the paragraph on page 160, seems very unusual since it ignores the existence of an elastic limit in dealing with materials under stress ; also, further on, it is stated that by giving the triangular wall a base of such length as to cause the resultant pressure on the base to pass through the down-stream edge of the middle third, a factor of safety of 'two' is secured ; because the moment of the weight of the wall about the down-stream toe is double the value it would need to have if the resultant pressure passed through that toe. Now, in the latter case the wall would be on the point of overturning, and a (theoretically) infinite intensity of pressure would be produced at the toe, instead of a pressure merely double the maximum intensity occasioned in the first case ; that is, the use here made of the term 'factor of safety' seems peculiar and misleading, since we generally consider the factor of safety to be the ratio of the breaking stress to the highest permissible actual stress. With the usual definition of the factor of safety, *eq.* (13) and *eq.* (14) of page 161, are evidently not in accord."

Much discussion has been evoked on the subject of founding bridge piers, as affected by the buoyant effort of the immersed masonry ; and Mr. Francis' ingenious apparatus does justify the expediency of fully taking into account the complete permeability of any dam or pier as a

useful precaution to offset the undiscoverable defects of building, when we are confronted with the impossibility of even guessing at the extent of such buoyant effort. But it cannot be admitted as a certainty that "an upward pressure may be transmitted through the mortar to the entire base" of the dam, for such a pressure could be transmitted only when the entire base is lifted from its support; and in this case no pressure could be exerted on the bed of the foundation. So long as the specific gravity of the material of the dam exceeds that of the water under the ordinary conditions of dams, direct contact and pressure must exist at many points of the base, however irregularly distributed or restricted they may be to small and unconnected areas. This fact points to the necessity of providing for increased surfaces, to diminish the probability of overloading the points of contact, thus increasing the probability of making them more numerous.

Experiments are now under way at the cement testing laboratory of Cornell University to ascertain the change in the law of pressures due to change in the linear dimension of the specimens subjected to compression; and with other valuable results already obtained in reference to this important subject, the entire investigation will be submitted to the Society.

Regretting that pressing engagements prevent me from extending this discussion to many other interesting points in Mr. Francis' valuable contribution, I will close these hasty notes by referring those who may wish to follow the argument of Mr. Francis on the subject of the formation of a plate-band in the material above the Chicago Tunnel, to Rankine's "Civil Engineer," pages 212 and 435. This subject is in a very unsatisfactory condition, and the novel aspect (to me) in which Mr. Francis presents his views, may open a new avenue for attacking this obdurate problem. I can see that in very solid rock a certain thickness above the top of a tunnel may act like a built-in beam resting upon the span made by the excavation; but in clay (more or less plastic) a more or less completely closed arch must be formed.

John W. Hill, M. Am. Soc. C. E.—In low dams, if the profile and weight of the masonry is sufficient to prevent sliding on the base, and overturning, there will be no danger from excessive pressure at the outer toe. In high dams, however, the conditions of stability and resistance to overturning may be had with a given profile and weight of masonry, but with dangerous pressures on the outer and inner toes; hence, the advantage of the Debauve method, quoted by Mr. Francis, which makes the test of strength, the pressures developed at the toes of the structure, in calculating the stability of high dams. If the maximum pressure at the base, or at any joint, *i. e.*, the pressure at the outer toe with the reservoir filled, or at the inner toe with the reservoir empty—falls within a limit (to be fixed), for the materials used in construction of the work, all other conditions for safety are necessarily fulfilled.

It has long seemed to the writer that the problems associated with masonry dams, not subject to a flow over the crest, were less complex than with dams subject as the Vyrnwy Dam for the Liverpool Water-works to periodical overflow; or as low dams placed in the channel of a river, subject to complete submersion. In the former the strains are divided into; (1) the pressure on the inner toe, with the basin or reservoir empty, and, (2) the pressure on the outer toe, with reservoir filled for high dams; and (1) the resistance to sliding on the base, and (2) resistance to overturning with water at the crest for low dams; while with dams constructed for an overflow over the crest, or submerged at flood, we have these strains to consider in addition to the variable pressure upon the face of the masonry, developed by the approaching flood, and the overflow of variable depth on the crest.

During the past season the writer had occasion to make plans for a low dam, submerged at flood, to be constructed in the channel of the Blanchard River, for the Findlay, Ohio, Water-works. The purpose of the dam was to create a storage of water in the channel of the stream above and in a reservoir adjacent to the river, about 1 000 feet up-stream from the dam. The reservoir is an artificial work, constructed in a natural depression, and the channel of the river immediately above the dam will be widened, by excavation of the soil of the banks to the natural rock (which constitutes the bed of the stream), for a distance of 1 500 feet above the dam. The slope of the bed of the river is 3.159 feet per mile, and the influence of a dam 8 feet high is felt for a distance of 2.53 miles up-stream. The storage provided in the artificial basin and the channel of the river is nearly 90 000 000 gallons.

In designing the dam, that is, in determining the heighth and length of overfall, two general objects were kept in view; (1) to produce the largest storage of water consistent with the safety of property above, and (2) to provide for the discharge of the flood volume, with the least increase of surface elevation of the water over the dam. The natural channel of the river, and the topography and value of the property at and near the dam site, and the cost of the structure, of course, were always modifying factors.

Calculations based upon the best obtainable rainfall data, the area of the water-shed of the stream above the point where the dam is located, and the topographical and geological character of the drainage ground, placed the ordinary flood discharge at 9 180 cubic feet per second. The overfall was accordingly made 200 feet long, and to the coping of the wing-walls, or roots of the dam, the depth was made 6 feet. That is, the clear water-way over the crest and between the wing-walls is 200 x 6 feet. The coping stones on the crest of the dam are bush-hammered on the upper beds, and inclined—with the lower edge up-stream—one foot in eight.

The clear water way of 1 200 square feet, within the wings of the

structure, according to Mr. J. B. Francis, has a flood capacity of

$$Q = 3.012\ L\ h^{1.53}$$

where L = length of overfall,
h = head over crest of dam ; and
Q = discharge in cubic feet per second ;
hence $Q = 3.012 \times 200 \times 6^{1.53} = 9\ 342$ cubic feet ; or slightly more than the assumed flood.

The waste way over the crest of the dam is not proportioned to an extraordinary flood, for from high water marks found along the river, and information obtained from farmers living for many years upon land adjacent to the stream, the writer assumed the extraordinary flood, which occurred February, 1884, as roughly 20 000 cubic feet per second, or more than twice the ordinary flood discharge.

The highest water mark (near the site of the dam), above the lowest elevation of bed rock in the channel is 17.96 feet ; the cross-section of the natural channel at an elevation of 8 feet (crest of dam) above same datum was 946.48 square feet ; the surface width of stream, at extraordinary flood tide, was taken as 500 feet, and the increased elevation of high water, with the obstruction created in the channel by the dam, was taken as 1.89 feet. The cross-section of the improved channel of the stream to the crest of the dam is 1 300 square feet, and the section of clear water way above the coping stones of the wing walls was taken as 1 400 square feet.

From these data the unbalanced pressures on the face of the dam—per foot of length—were calculated as follows :

Water at crest	2 000 pounds.
Water 6 feet over crest	4 582.2 "
Water at extraordinary flood, 11.85 feet over crest	3 990.0 "

The resistance to sliding, on the base, was calculated as follows:

$$R = \frac{c\ w\ f}{p}$$

where c = volume of masonry per lineal foot of dam, in cubic feet = 78.25;
w = weight in pounds of masonry, per cubic feet = 144;
f = co-efficient of friction, taken as 0.60;
p = unbalanced pressure, in pounds, on the face of dam = 4 582.2; and
R = resistance to sliding, in terms of unbalanced pressure (pressure taken as unity), factor of safety.

Inserting values: $R = \frac{78.25 \times 144 \times 0.6}{4582.2.} = 1.475.$

The bed of the river is solid rock (limestone) of the Niagara series, nearly level from end to end of the dam, and is cut, as shown by the accompanying sketch, Plate XXII, to receive the footing courses, and the resistance of the structure to sliding, instead of being 1.475, based on smooth continuous surfaces, is really equal to the cohesion of the rock in the bed of the stream and the dam.

The resistance to overturning, in terms of the unbalanced pressure on the face, was calculated as follows:

$$L = \frac{c \ w \ x}{p \ y}$$

where x = horizontal distance, in feet, from center of gravity of section of dam to outer toe = 7.2;

y = vertical elevation, in feet, of point of application of pressure to face of dam = 5.03;

L = resistance to overturning.

Inserting values:

$$L = \frac{78.25 \times 144 \times 7.2}{4582.2 \times 5.03} = 3.52.$$

Testing by the Debauve method:

$$p' = \frac{2\,(2l - 3u)\,P}{l^2}$$

where l = length of joint, or width of base = 12 feet;

u = distance from outer toe to point of application of pressure, p = 5.17 feet;

P = resultant of vertical and horizontal forces = 11 268 pounds; and,

p' = pressure in pounds per square foot at outer toe.

Inserting values:

$p' = 2\ [2 \times 12 - (3 \times 5.17)]\ 11\,268 \div 12^2 = 1\,328.685$, or 9.227 pounds per square inch.

There is one element in the construction of high masonry dams, and perhaps other structures, which Mr. Francis does not mention, viz., the importance of a solid, unyielding foundation. Even if the profile be correct, and the pressures at the toes of the base be within the limits given by the Debauve method (92.5 to 114 pounds per square inch), settlement due to the vertical component of the forces developed at the toes may ruin a structure of this character.

The lower silurian formations, and often other series of stone in parts of the country, are in layers or ledges separated by seams or strata of clay, which, when moist, will yield and flow almost like a lubricant; and before placing an important work like a high dam or retaining wall, upon such a foundation, careful borings and drillings should be made in the subterranean formations. Should the strata of the foundation be separated by seams of clay, the probability of its becoming softened and

forced out under pressure is necessarily increased, when the structures over it are a high masonry dam and a deep reservoir of water.

A settlement which would have no dangerous effect on the foundation of a building, might threaten the integrity and usefulness or even the safety of a high dam.

The main retaining wall of the Eden Reservoir of the Cincinnati Water-works failed from this cause. This wall has a height at the center of the ravine across which it is placed of 118 feet and a width of base of 48.5 feet, and is cracked on the face in several places, the cracks having a generally vertical direction, and sensibly widening from the top downwards. This wall is built of a compact limestone, cost nearly $800 000, and was built in the usual "happy-go-lucky" manner of city works.

Taking the highest limit assigned by M. Debauve, or rather the pressure developed at the outer toe (with reservoir filled), for the Ban (France) dam, the load on the foundation becomes $113 \times 144 \div 2\,000 = 8.14$ tons per square foot, while M. Krantz, chief-engineer of bridges and highways, France,* puts the limiting pressure or load per square foot at the toes of high dams at 6.12 tons, and after recounting several disasters with high masonry and other dams, says: "In view of such chances, the engineer must not be too bold, nor take upon himself before the public a responsibility which is powerless to repair such disasters. However little it may lean toward rashness, boldness in such cases may become almost a crime. * * * * In my opinion it is better not to build reservoir walls if we have not the necessary resources to build solidly, than to build carelessly at the risk of frightful catastrophes. On all these accounts, I think that in calculating strength the pressure of 6 kilogrammes per square centimeter (85 pounds per square inch), should never be overstepped."

Mr. George F. Deacon, borough engineer of Liverpool, however, upon investigating the plans for the Vyrnwy dam, found the pressure at the inner toe, with reservoir empty and a wind pressure on the back, of 40 pounds per square foot, to be 8.7 tons; and with reservoir full, and a wind pressure of 60 pounds per square foot on the exposed face, a pressure or load of 6.36 tons on the outer toe.

The foundation of this structure is "on rock belonging to the Caradoc group of the lower silurian system." Mr. Deacon, in his report to the corporation, says the dam "has an exceptionally high factor of stability."

The facility with which water under moderate pressures will percolate through brick, sandstone, unglazed stoneware and porcelain, is taken advantage of in the construction of our best known domestic filters. The "Pasteur" filter is constructed with tubes about 1-inch in-

* "Study on Reservoir Walls," by J. B. Krantz, Ingénieur des Ponts et Chaussées, Paris. Translated by Captain F. A. Mahan, U. S. A., 1883.

Masonry Dam
Findlay O. Water Works

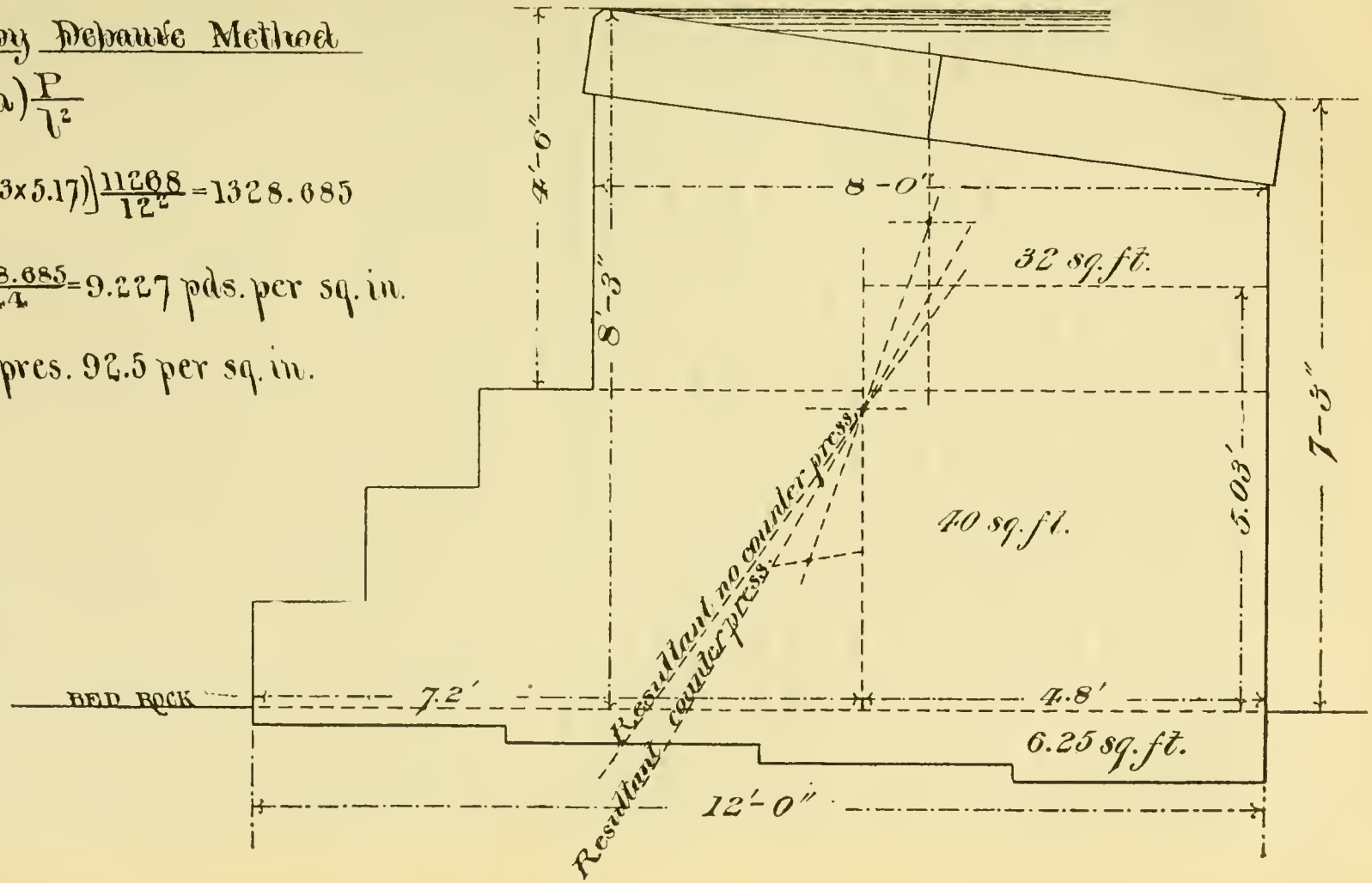
Stability by Debauve Method

$$p = 2(2l - 3a)\frac{P}{l^2}$$

$$p = 2\left[2\times 12 - (3\times 5.17)\right]\frac{11268}{12^2} = 1328.685$$

and $\frac{1328.685}{144} = 9.227$ pds. per sq. in.

Limiting pres. 92.5 per sq. in.



ternal diameter and $\frac{3}{32}$ inch thick, of unglazed French china clay, and under a pressure of 25 pounds per square inch, seven such tubes, about 9 inches long, will filter all the water required for drinking and cooking purposes by a family of eight or ten persons.

The writer has known of instances of earthen reservoirs, the inner slopes of which were paved with two courses of hard burned brick, set in cement mortar, leaking badly, even after the application of successive courses of strong cement grout over the face of the brick work. An engineer, well known in the water-works line, after constructing a small reservoir, in which the puddle was laid on a slope of one and one-half to one, and carelessly placed and tamped, attempted to make the basin water-tight by a layer of cement over the brick paving, with no success. The paving in this instance was in two courses, the lower of bricks, set flat on a bed of sand, and the upper of bricks on edge, set in neat Portland cement.

In Mr. Francis' experiment upon the perviousness of cement mortar under pressure, it is to be regretted that his tests were limited to one thickness and one pressure; tests made on different thicknesses of mortar, or cement slabs, and at different guaged pressures, would have furnished the data for an expression showing the relations of pressure, thickness, and rate of percolation. Cisterns built of brick in cement mortar, are never proof against infiltration of water from the surrounding soil, and thin foundation walls of buildings, if the soil be damp, are frequently covered with a sweat upon the inside. Thick walls, of well built masonry, when the thickness is 5 feet or more, are often dry upon one side while in contact with water upon the other; which suggests that there is a minimum thickness for certain materials, through which water will not pass by percolation, even under considerable pressure. This thickness, of course, must increase in some ratio (unknown to the writer) with the pressure.

Mr. Fteley.—In connection with the proposed construction of the Quaker Bridge Dam for the additional water supply of New York—a structure of unusual magnitude—opposite opinions have been expressed by engineers in regard to the advisability of adopting a straight or a curved line for the plan of high masonry dams. Any remarks on this subject would, without doubt, be received with a great deal of interest.

Mr. Francis.—It is a matter I have given some thought to, but have made no calculations upon. We can test it, however, by the consideration of extreme cases.

First.—If the dam is built in the form of an arch, and depends for its stability entirely upon the transfer of the pressure against it to abutments, similar to an ordinary bridge; when the pressure is brought against it there will be a yielding corresponding to the settling of an

arch when the centers are struck. If the dam is long, say a thousand feet, the amount it will yield will obviously be considerable; perhaps, in the middle in a high dam, a foot or more. Being built on the rock, which cannot yield laterally, this would cause a sliding on the base or fractures in the masonry, most probably the latter, which would be a fatal objection.

Second.—If the dam has a section sufficient to resist the pressure of the water at every point without yielding, independently of lateral support, if built in the form of an arch it can get no additional strength from it, as the support due to the arch cannot come into play until there is some yielding to the pressure.

According to my view, the two forms of resistance are not homogeneous and cannot be made to act harmoniously together.

N. M. EDWARDS, M. Am. Soc. C. E.—I should think that in large masses of masonry in high dams, the question of cracks would be important in determining that the dams be statically strong.

In fissures, either great or small, hydrostatic pressure would have its power in proportion to their extent, and tend to disintegrate and destroy the structure.

Even if it were possible to build it in a solid mass, a crack or cracks from a fault in the earth's crust, might enter in, and thereby the masonry be resolved into a broken mass.

Mr. FRANCIS.—The experiments on the increased resistance per square inch to crushing depending on increase of surface, which I have quoted from General Gillmore's experiments made in 1875, were limited to hand specimens by the limited capacity of the hydraulic press with which they were made, which did not exceed fifty tons. The experiments have since been extended with the testing machine at the Watertown Arsenal, near Boston, which has a capacity of four hundred tons. The results of the experiments made with this machine on cubes up to twelve inches on the side and other forms, and of various materials, have been published during the current year.* They do not bear out the results of the great increase of resistance per square inch to compression deduced from the previous experiments on the smaller cubes, the accuracy of which is, however, not questioned. The causes of the differences are explained by General Gillmore in his summary, as follows :

"The law of increase of compressive strength per square inch of bed-surface, with increasing size of cubes, which was based upon experiments made some ten years ago with various but limited sizes of Berea sandstone, was not confirmed when larger cubes of Haverstraw

* Notes on the Compressive Resistance of Freestone, Brick Piers, Hydraulic Cements, Mortar and Concretes. By Q. A. Gillmore. New York: John Wiley & Sons, 1888.

sandstone, cement, mortars and concretes were tested. That some such law exists for cubes within certain limits, cannot be doubted, not only in view of the Staten Island experiments, but of experiments made by foreign investigators, referred to in this report. The failure of the law with larger cubes seems to be due to the lack of the homogeneity throughout the mass of such pieces; this is indicated by the strain diagrams. It is only possible to discover defects in a large piece by dividing it into smaller pieces; and when the most perfect of these fragments are selected to prepare small test-samples, approximately true unity in regard to homogeneity of structure may be obtained. It is thought that large cubes are not such units, or true monoliths; that they represent a species of conglomerate of smaller irregular pieces, bound together by a cementing substance of varying strength, and perhaps partially separated by minute cracks and cavities. With cements, mortars and concretes, the relative amount of work expended in consolidating the material in the moulds cannot well be evenly distributed or proportioned for all sizes of cubes; the amount of set developed in small and large cubes of the same age is undoubtedly different. This is probably the reason why in all of the cements, mortars and concretes the smallest sizes of each series of cubes proved the strongest per square inch of surface pressed."

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THE CONSTRUCTION OF THE SWEETWATER DAM.

By JAMES D. SCHUYLER, M. Am. Soc. C. E.

READ OCTOBER 17TH, 1888.

WITH DISCUSSION.

The question of an adequate water supply for irrigation, as well as for the domestic use of cities and towns, is one which, in San Diego County, California, necessarily involves storage reservoirs. The streams of the county are of an intermittent character. The mountain ranges in which they head and from which they flow to the coast do not generally exceed 6 000 to 6 500 feet in elevation—an altitude too low in the latitude of San Diego to maintain perpetual snow upon their summits, or even to retain such proportion of the winter precipitation as comes in the form of snow (not usually more than ten per cent.) for more than a few days or weeks. As a result the streams are torrential in winter and carry large volumes of water, but in summer and fall, when most needed for irrigation, are almost dry for twenty or thirty miles of their lower course, with the exception of certain seasons of such unusual

rainfall that no irrigation is required—seasons that come at rare intervals. Ordinarily the streams in summer reverse the usual order of nature, and are largest at the small end, and to get a water supply the engineer must either go far back into the mountains and gather together a number of small living streams and springs, and pipe them out long distances, or construct dam and storage reservoirs to retain the winter floods. Fortunately nature has compensated for the existing conditions by providing numerous favorable sites for such constructions, and every stream of importance in the county has available sites for storage dams of large capacity. A number of water companies are engaged in preparing for extensive works of this character, which, when completed, will provide irrigation facilities for several hundred thousand acres of land otherwise unproductive.

This era of development was inaugurated but recently, and the first completed work of the character is the Sweetwater Dam and Reservoir and the extensive pipe system reaching out from it.

The circumstances which led to the building of the dam were that the San Diego Land and Town Company (a first cousin of the Atchison, Topeka and Santa Fé Railway), owned a large body of fertile and desirable mesa and valley lands bordering on San Diego Bay, adjacent to San Diego on the south, which were unsalable without water to irrigate them. These lands constitute the greater part of the "Rancho de la Nacion," including the town site of National City, which also languished with thirst. The Sweetwater River passes nearly through the center of the lands, and is of the nature described—intermittent in flow, at least for many miles above its mouth.

The first storm or two of the rainy season is absorbed by the thirsty earth, and the stream generally does not begin flowing into the bay until late in December, or in January. After each heavy storm thereafter its volume will reach 500 to 1 000 cubic feet per second, for a few days, and within a fortnight recede to 10 cubic feet per second.

The last severe storms of the rainy season are usually in March, and the flow of the stream will generally dwindle to one or two cubic feet per second by June 1st, which amount may be maintained through the remainder of the year, but not always. The large supply running to waste each year, followed by months of scarcity, naturally suggested storage, and the first cañon above the mouth of the stream was selected as the place to accomplish the object. This narrow gorge, 7 miles east

Plate XXIII.—Sweetwater Dam.

of the bay, is a deep and narrow cut, half a mile in length, through a dyke of trap rock or trachyte that intersected the valley of the Sweetwater, leaving above it a broad level valley some 3 miles long, ¼ to ¾ mile in width. This is the site of the reservoir formed by the dam built at the head of the gorge.

The construction of this dam was decided upon and work begun in November, 1886. The original plan designed was a narrow wall of concrete masonry, 50 feet high, 10 feet wide at bottom, 3 feet on top, arched up stream. On the upper side an embankment of loose earth was to be filled in against the masonry wall to its full height. After two months' work had developed the character of the design, the plan was disapproved by the management, and the writer was called upon to design a suitable structure and execute its construction. Some thirty-five thousand dollars had already been expended, and in order to utilize as much of the old work as possible, the new structure was planned to rest upon and encase the foundations already laid. This decision influenced to some extent the radius of the arch of the new dam, as well as its position relative to the axis of the cañon and the location of the auchorage on the sides. In other words, to avoid throwing away the work already done, the new work was adapted to the old in a way that ultimately increased the length of the dam on the crest somewhat more than would have been necessary by shifting the point of radius to one side of the central axis of the cañon, and making the radius somewhat shorter than it otherwise would have been. An engineer is sometimes driven to adaptations of this sort against his judgment, to save, or to give the appearance of saving, the pockets of his employers.

The modifications of the original plan were radical ones. The combination of earth and masonry was rejected, as it seemed to the writer that water alone was sufficiently heavy for the masonry wall to support without adding the last straw on the camel's back, of a mass of saturated earth. A gravity profile was adopted, and rubble masonry formed of blocks of stone up to four tons weight, was substituted for bastard concrete composed of cement mortar, with small stones rammed into it, which had been previously used. So much of the old plan was retained, however, as to form an embankment 50 feet wide on top, 10 to 15 feet high across the cañon, against the face of the wall, but clay, well rammed in layers, was substituted for the silt and quicksand loosely dumped, with which the dam was formerly being made. The object of

this clay-filling was to cut off possible seams in the bed rock underneath the dam, and reduce the pressure on the structure. The top of the embankment is 70 feet below the top of the dam.

The Foundation.—After the bowlders, sand and gravel had been stripped from the base of the dam on either side of the old work, the bed rock was found to be very irregular in surface, presenting the appearance of a number of pyramids and cones thrown heterogeneously together, but bound solidly in one mass, and well polished by attrition. The rock was very close in texture and exceedingly hard. No attempt was made to cut out the bed in level benches, as the unevenness of the bottom, as nature left it, gave the assurance that whatever movement might occur in the structure built on such a base, there could be no possibility of its slipping or sliding on the base. Wherever there were seams in the rock they were invariably occupied by roots, and the excavation was carried down till the seams pinched out and the roots disappeared. The rock was then thoroughly scrubbed by hand, and a thin grout of pure cement applied with brooms, filling the minutest crevices and angles in the rock, before starting the masonry.

The side walls of the cañon required more excavation to reach a satisfactory anchorage than the bottom. The north side was composed of shattered rock scored with innumerable seams, filled with red clay. In this material the excavation was carried to a depth (perpendicular to the slope) of 20 to 25 feet, before a solid ledge, free from seams, was encountered. This ledge lay with a slope nearly parallel with the surface slope, and in direction so nearly parallel to the radial line of the curve of the dam, that it could not have been better placed to receive the arch thrust, and formed a natural skewback. This was carefully stripped and treated with cement grout in the same manner as the base.

The abutment on the south side was against the end of a dyke of trap rock, crossing over the hills to the south in a direction nearly parallel to a line passing through the center of radius, and dipping westward at an angle of about 10 degrees from the vertical. After cutting into the face of this rock 5 to 10 feet, all seamy, loose material was stripped away, and a bedding that was deemed sufficiently good was obtained, although the rock was not as free from seams nor as solid in mass as the north abutment. However, the entire foundation is an admirable one, of rock in place throughout.

Plate XXIV.—Sweetwater Dam.

The Plan.—The original height of 50 feet was arbitrarily adopted at the beginning of the work, without any special investigation of the quantity of water to be stored by a dam of that height, but was "guessed" to be sufficient for present necessities, and the estimate of its cost was considered to be about the limit of the expenditure the company cared to make on an experimental scheme. There was an immediate and pressing need for water, the rainy season was passing, and it was desired to get up a part of the structure as rapidly as possible in order to catch a partial supply for the coming summer. Accordingly, in compliance with this desire, the foundation was rapidly laid and the structure hurriedly carried up to a point where it was safe to begin catchment. The base of the dam was laid with a width of 36 feet, and at a height of about 15 feet above the lowest course, it was drawn in to a thickness of 24 feet. At this level (whose elevation above tide is 140 feet) the lowest pipes pass through the dam. Above this level the structure was carried to a height of 45 feet, with a top width of 5 feet, base as stated of 24 feet, face batter (upstream) of 1 to 6. In anticipation of a probable addition to the height of the dam in future, the back was built in three steps, to give an opportunity of bonding the new work to the old. The profile of this portion of the structure is shown in Plate XXX. It was a gravity profile, whose line of pressure passed within the inner third of the base. It was constructed in arch form, convex to the stream, on a radius of 225 feet on the face line at top.

During construction the stream was carried in a conduit 30 inches square through the masonry near the bottom of the original creek bed. But one storm of the season of 1886–87 (a dry one) swelled the creek sufficiently to exceed the capacity of this conduit, and then it rose and ran over the top of the masonry for two days only, without injury. This occurred February 14th and 15th, 1887, when the flow reached a maximum of about 500 cubic feet per second. The gate at the upper end of the conduit was finally closed April 20th, 1887, and the conduit was filled solid with masonry from below. From that time until June 1st, the catchment was about 80 000 000 gallons.

By the 1st of June the structure, as planned, was completed to the height of 60 feet above the bottom, 10 feet higher than the height originally contemplated. It contained about 7 500 cubic yards of masonry and had cost all told (including the preliminary experiments) about

$100 000. Meantime, surveys of the reservoir basin and watershed had developed the fact that the 60-foot dam would impound 1 221 million gallons, whereas, its extension to 90 feet in height would give a capacity of nearly five times that quantity, or 5 882 million gallons. Also that the area of the watershed tributary to the dam is about 186 square miles, of which one-third is above an elevation of 3 000 feet, and between that elevation and 6 500 feet. The watershed was evidently ample to justify the hope that the greater reservoir would be filled almost every year of ordinary rainfall. The increased volume of water stored would so largely extend the utility of the works, and give so considerable increase in security against the disasters following a severe drouth, that the increased expense of extending the height of the dam while the working force and plant were on the ground and fully organized, seemed to be immediately justifiable. These arguments were embodied in a report, which was favorably considered by the directors of the company, and orders were given, about a fortnight before the 60-foot dam was completed, to extend the structure to a height of 90 feet.

This somewhat extended account of the growth of the enterprise from small beginnings is necessary to an understanding of the causes that led to the building of the structure in two sections rather than as a mass. The fact is that the work was nearly half done before all the conditions were thoroughly understood—conditions which ordinarily in works of such magnitude and importance are known, studied and exhaustively discussed preliminary to the beginning of any work whatever.

In designing the plan of the higher structure, greater reliance was placed upon the arched form than in the lower dam, then approaching completion. The profile adopted was one which theoretically gave stability by its own gravity, but without as large a factor of safety. The line of pressure falls about the center of the lower third. It was reasoned that as the foundation was as near perfection as can be generally found, apprehension on that score was unnecessary—and the source of the failure of such of the great masonry dams of the world as have given way—insecure foundations—need not be regarded as a factor in this case. If one can imagine a monolith to be carved in the form of a true arch, of such weight and dimension that any section of it is capable of withstanding the pressure of quiet water against it to its full height, without sliding or overturning, and such a monolith be firmly wedged

between the rock-bound walls of a narrow cañon, the possibility of its being ruptured, displaced or destroyed from water pressure alone, cannot readily be conceived. Now, if by the use of rich cement mortar and the best of building stone a structure be formed of the same dimensions and in the same position, which in time becomes virtually a monolith, based on the firmest of bed rock, its stability must be equally assuring to the mind.

The dimensions adopted were the following: base, 46 feet; top thickness, 12 feet; height, 90 feet; radius of arch, 222 feet on line of face at top. The face batter of 1 to 6 was carried to within 6 feet of the top; thence to the top of the parapet wall, vertical. The batter on the back started at the top with 1 to 3 for 28 feet; thence 1 to 4 for 32 feet; thence 1 to 6 to the coping.

The Construction.—When the new work was begun at the base of the completed structure, special care was taken to secure a perfect footing for the toe. When the foundation was stripped it was found that there was a slight leakage at various points along the bottom of the masonry, amounting altogether to about 10 000 gallons daily. The only perceptible leakage through the masonry was along the sides of the waste conduit, which had been recently filled in, although there were moist spots all along near the bottom.

All the leakage was entirely cut off by the new work, although it was necessary to carry up small well-holes alongside the old masonry to within about 15 feet of the level of the water surface in the reservoir, and keep them pumped out, before it was safe to close them entirely. Water was standing in the reservoir at a height of about 35 feet above the base of the dam, and the small quantity of leakage, and the ease with which it was stopped, was considered a favorable test of the superior quality of the masonry.

The stone used was of two grades, a dark blue and a gray metamorphic rock, impregnated with iron. The gray stone is full of minute quartz crystals, and is of slightly less specific gravity than the blue stone, which carries more iron. It was obtained from a quarry opened in the face of a vertical cliff over 100 feet high, situated 800 feet below the dam. It has no well-defined cleavage, and broke out in irregular masses, although generally having one or more tolerably smooth faces. Numerous tests of its specific gravity gave its weight as 175 to 200 pounds per cubic foot. The average weight of the masonry in place was

estimated at 164 pounds per cubic foot, which was the value used in the calculation of stability and strains.

Portland cement of the best obtainable quality was used in the proportion of one part of cement to three parts of sharp river sand. For the upper 4 feet next to the water a richer mixture of one to two was used. The sand was clean, sharp, and of the most suitable degree of coarseness to make the best of mortar.

In all the later portion of the work, from May 1st till its final completion, the mortar was mixed in a machine invented and patented by Mr. E. L. Ransome, of San Francisco, consisting of a cubical dice-box suspended on bearings attached at two corners diagonally opposite, through the center of which passed a perforated tube for injecting water, the box being revolved by horse-power. The ordinary charge was three barrels of sand and one barrel of cement, which was dumped into a hopper from a platform above the mixer, and admitted into the box through a door. The box was generally given three or four revolutions after charging with sand and cement before the water was admitted. A cock from a small tank regulated the flow of water, which was turned in slowly, the whole supply required being admitted in the next three or four revolutions. Eight to ten revolutions were sufficient to thoroughly mix the mortar, requiring two to three minutes in all. The batch, when mixed, was dumped into a box with a hinged bottom, resting on a low car, and run out on a tramway within reach of the derrick standing on the wall. Four chains of equal length, attached to the corners of the box with rings at the end, served to hoist it from the car, and the derrick delivered it where required on the wall, the latches holding the bottom in place were tripped, and the load instantly dumped within reach of a group of three or four masons.

After the completion of the 60-feet structure the tramway for delivering mortar was carried entirely around the face of the dam on a bracket trestle suspended from bolts driven into holes drilled in the wall, the track being on a level with the top. A grade of 3 feet in 40 at the end next the mixer was sufficient to give impetus to the car and carry it to the farther end of the dam, and a brake was attached to stop it where required. This tramway did excellent service, and remained in the same position until the dam was carried to its ultimate height, 30 feet above the level of the tramway. Prior to adopting this device the mortar had been mixed by hand, and delivered by hod carriers. With

Plate XXV.—Sweetwater Dam.

the mixer and tramway, five men and a horse performed the work formerly requiring sixteen to twenty men (four mixers and twelve to sixteen hod-carriers), more promptly and satisfactorily. The hoisting of the mortar by the derricks was performed without interference with their regular work.

The stone was hauled on wagons, rigged up with platforms on a level with the hind wheels. The derricks in the quarry were simple shear-legs, slightly inclined from the vertical. The stone was hoisted under the center of the shear-legs, the wagon driven underneath it, the load gently lowered, and the chain left in position for use in hoisting at the wall. Stones weighing 6 000 pounds were not infrequent—and even greater ones were readily handled on the wagons and hoisted into position. All stones smaller than 500 pounds weight, were loaded on stone-boats, 4 feet square, made of 3-inch plank with a bottom of boiler plate. These were provided with chains at the corners for hoisting. They cost $30 each; were made very strong, and yet the wear and tear was so great that several sets of them were worn out on the job.

All the hoisting was done with horse-power. This was frequently criticised as questionable economy, but taking into consideration the scarcity and high price of fuel, the cost of the number of hoisting engines that would have been required, the delays occasioned by breakages, the skilled labor required to do the work, etc., the advantage in cost and convenience was on the side of animal power. The writer had a previous experience during the construction of a section of the San Francisco sea-wall, which led to this preference. Steam and horse-power were there applied to the derricks in the quarry, side by side, in the handling of over 100 000 tons of rock, and the greater economy and convenience of horse-power was marked and decided.

Four derricks were in constant use on the wall, and at one time five. The masts of four of them were 30 to 38 feet in length, the booms 26 to 32 feet. A larger one with mast 50 feet, and boom 45 feet, was so superior in efficiency to the others, that it was a matter of regret that all had not been of larger size. All the work was done without serious accident or loss of life, although there were narrow escapes, and several horses were killed or injured. It was doubtless due to constant vigilance and inspection of apparatus, that no fatal casualties were recorded.

The work was completed April 7th, 1888, having occupied sixteen months in construction, including two months of delay on account of shortage of cement.

The total volume of masonry laid was as follows:

In the dam proper	19 269.0	cubic yards.
" wasteway	481.2	"
" inlet tower	376.8	"
" conduit from dam to tower	182.0	"
" gate houses	71.0	"
In various accessories	127.0	"
Total	20 507.0	"

In this work 17 562 barrels of cement were used, an average of 1.17 cubic yards of masonry per barrel of cement.

The total cost of the dam was $234 074 11, distributed as follows:

PLANT.—Tools, etc		$6 236 76
MATERIALS.—Cement	$63 111 03	
Cement hauling	8 614 18	
Lumber	2 408 08	
Iron work	4 915 99	
Pipes, gates, etc	5 152 58	
Miscellaneous materials, powder, etc	3 229 84	
		87 431 70
LABOR.—Common and skilled labor	$93 590 55	
Foremen	6 866 49	
Teams	19 696 12	
Engineering, salaries and expenses	10 555 20	
Clerical work	653 88	
Earth work (contract)	7 666 51	
Miscellaneous expenses	1 376 90	
		140 405 65
Total		$234 074 11

The cost of the flowage tract for the reservoir is not included in the above. A little over one-half the land cost $16 426 93. The remainder is in litigation under an action of condemnation. A San Diego jury under the stimulus of "boom" prices awarded the owner $280 an acre, or a little over $100 000 for land, one-third of which was worthless, and the remainder unimproved. This judgment is being contested before the Supreme Court. The clearing and grubbing of about three hundred acres of the reservoir basin cost $10 808 46.

Measurements were taken weekly of the volume of masonry laid, and these measurements compared with the expense account, as a check upon the cost of each department of the work, and the system thus maintained throughout served to point out reforms needed from time to time. The average cost per cubic yard of 11 322 yards of masonry laid during the period from May 1st to December 31st, 1887, is shown in the following table:

Items.	Cost.	Cost per Yard of Masonry.	Percentage of Total Cost.	Remarks.
Cement—9 237 barrels at $4.20....	$38 795 40	$3.427	38.900	Average 1.22 per bbl.
Stone—Quarrying.....	4 816 23	.425	4.829	
Loading.....	5 917 83	.523	5.933	
Hauling.....	4 756 75	.420	4.768	
Hoisting.....	6 531 52	.577	6.550	
Sand—Loading and hauling.....	3 902 61	.345	3.915	
Mortar—Mixing and delivering .	2 702 27	.239	2.710	
Masons.....	9 025 05	.797	9.050	
Helpers.....	2 101 30	.186	2.109	
Excavations of foundations.....	3 433 66	.303	3.444	
Making and repairing roads.....	1 332 30	.118	1.336	
Blacksmithing (labor).....	1 847 97	.163	1.854	
Carpentry.....	1 100 29	.097	1.104	
Rope.....	1 181 70	.104	1.186	
Tools.....	521 68	.046	.524	
Steel.....	153 59	.014	.155	
Blacksmith coal.....	107 48	.009	.109	
Blocks and sheaves.....	131 44	.011	.133	
Powder.....	971 00	.086	.974	
Foremen.....	3 763 11	.332	3.774	
Engineering and superintendence	3 880 00	.343	3.891	
Lumber.....	2 212 92	.195	2.220	
Wetting masonry.....	540 15	.048	0.542	
Total.....	$99 726 25	$8.808	100.000	

The Wasteway.—This important adjunct to the dam was carefully considered and proportioned to carry the probable maximum flow of the stream that may be presented for discharge, with a full reservoir. It is located at the south end of the structure, and is 40 feet in length by 5 feet in depth, divided into eight bays of 5 feet each. These bays are formed by piers of masonry, set at right angles to the flow, and provided with recesses on the upper face, in which loose flash-boards of 2-inch plank rest on an incline of 35 degrees from the vertical. Any set of boards may be removed from top to bottom, or the water may be held at successive levels from the top to the bottom of the weir by removing the top boards all the way across. The water falling over the weir drops

into a series of pools, 3 feet deep, which relieve the structure of shock, and passes down an inclined plane with a fall of 1 to 10, until it is carried away from the dam a distance of 50 feet, and then plunges into the cañon below. The capacity of the wasteway is about 1 500 cubic feet per second. This may be increased to about 1 800 cubic feet per second by opening a 30-inch blow-off gate in the main pipe below the dam.

The Inlet Tower.—This structure is located 50 feet above the dam, nearly opposite its center. It is built of masonry, with cement mortar mixed two to one, plastered outside and in with two coats of mortar mixed with one of sand to one of cement. It is 16 feet square at the base for a height of 10 feet, where its form is changed to a hexagon, with walls of a uniform thickness of 3 feet to the top. Each of the sides of the hexagon measure 3 feet on the interior face. Into the walls of the tower are built seven cast-iron elbows, at an elevation of 10 feet apart from bottom to top, the upper one being 10 feet below high-water line. The bell-mouths of the elbows open upward, and are ordinarily closed with a plain valve or cover of iron. The design is to draw water from the surface at whatever stage it may be. When any one cover is removed, a basket screen is lowered in its place, fitting closely into the mouth of the elbow. Three pipes pass through the dam and enter the tower at the bottom. The two lowest pipes are of cast iron, 14 and 18 inches in diameter respectively, and lie side by side. They are encased in concrete throughout, from the tower to the dam. On top of them is built a conduit of masonry with a circular orifice 40 inches in diameter, formed of walls 30 inches in diameter, in double arches. This conduit leads from the interior of the tower to the center of the dam, where it joins a pipe of $\frac{1}{2}$-inch boiler-iron, 36 inches in diameter, leading to the main gate immediately below the dam, and from this gate is carried the main pipe line down the valley. The smaller pipes are not at present used, except to supply a hydraulic ram throwing water to the keeper's house on the hill, 150 feet above, and to drain the tower when all the valves are shut. They are intended to be used for supplying a turbine and pump to throw water to a higher level than the dam will now reach.

As an illustration of the fact that masonry laid in Portland cement in the proportion of two of sand to one of cement, may be made water-tight with care exercised in laying, this tower, and the conduit leading from it, may be cited. When they are empty the pressure from the outside at present is somewhat more than 20 pounds per square inch on the

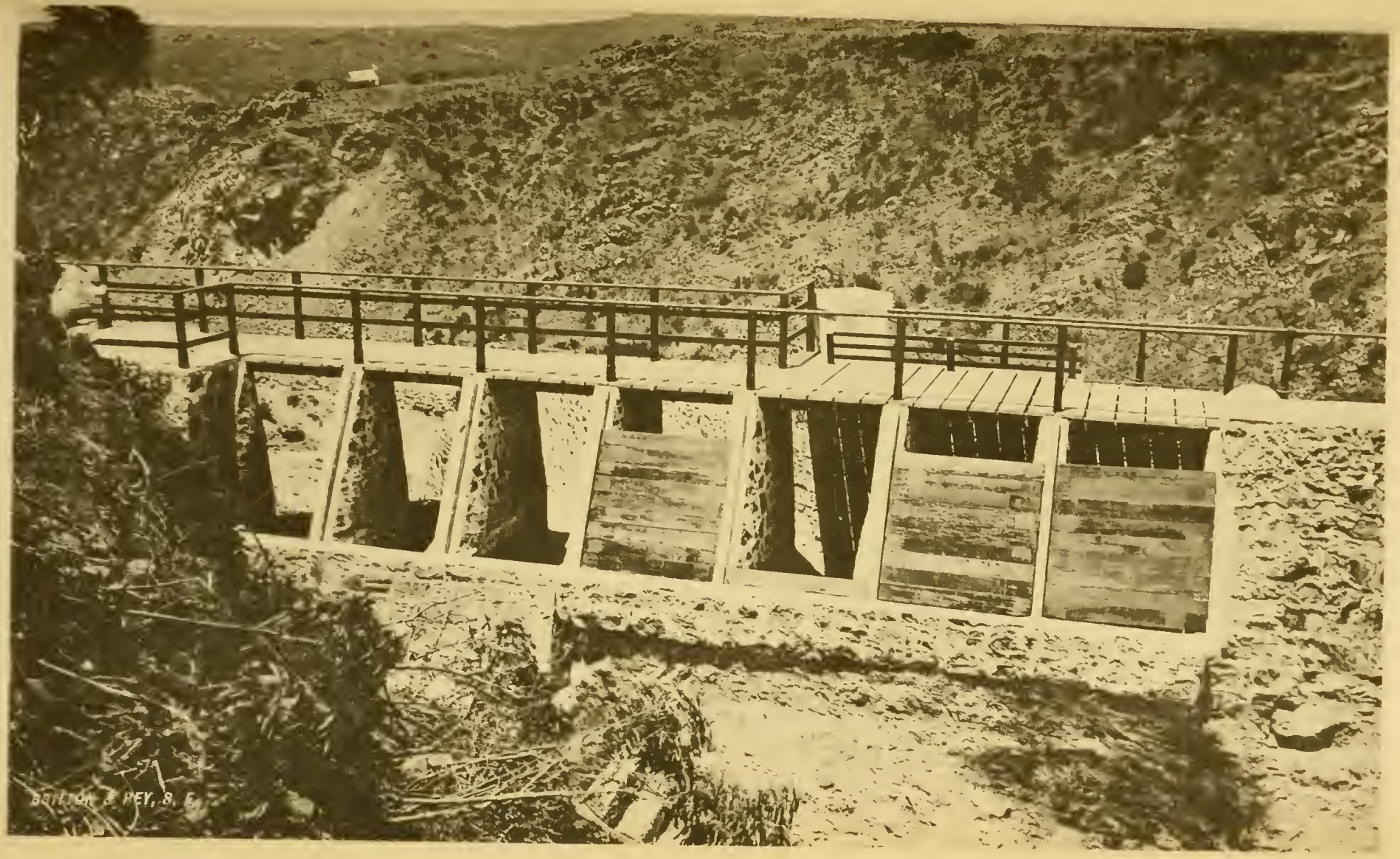


Plate XXVI.—Sweetwater Dam.

conduit and at the bottom of the tower, and there is no leakage in either of them.

The Reservoir.—Red clay soil constitutes the bed of the reservoir basin, or the major portion, outside of the old river-bed and bottoms, and is of an impervious nature. The following table of area and contents of reservoir is presented:

Contour elevation.	Area in acres.	Contents. Gallons.
145 feet (level of lowest outlet valve in tower)	3.51	
150 "	10.72	11 640 000
155 "	17.12	30 577 000
160 "	43.10	79 631 000
165 "	75.21	175 819 000
170 "	113.40	329 546 000
175 "	153.75	547 069 000
180 "	200.77	835 851 000
185 "	272.22	1 221 355 000
190 "	326.96	1 710 583 000
195 "	397.85	2 302 261 000
200 "	463.80	3 005 642 000
205 "	538.94	3 824 197 000
210 "	630.94	4 778 549 000
215 "	721.86	5 882 278 000

The table shows that eighty per cent. of the capacity of the reservoir is within the upper 30 feet of height, and that forty per cent. is within the last 10 feet. This fact reduces within small limits the fluctuation of head on the mains after the reservoir is once filled, and constitutes one of the reasons for increasing the height of the structure, as it enables the establishment of the probable limit of irrigation on the lands below, at a line not lower than 25 to 35 feet from the top of the dam. The irrigable area was thus largely increased, by reason of the decrease in fluctuation of depth in the reservoir.

A line has been drawn upon the map, Plate XXXI, which represents approximately the limits of the irrigable land directly commanded by the reservoir.

The cement used in the construction of the dam was principally such as could be obtained in the market of San Francisco, at a time of extreme scarcity. White's, Gillingham, Knight, Bevan & Sturge and Kaufmann & Lieffmann (German cement) were the brands chiefly used; but at times it was necessary to take inferior brands, and reduce the proportion of sand, to keep the work going. The average cost of the

cement laid down in San Diego was $3.66\frac{1}{2}$ per barrel; seventy cents per barrel was the price paid for hauling the greater portion from San Diego to the dam. Just as the work was nearing completion, a branch of the National City and Otaz Railway was finished to the dam, and a part of the cement delivered by that means at less rates.

Price of Labor.—Masons were paid $4 to $5 per day; common labor, $2 to $2.50; foremen, $4 to $6; carpenters, $3.50 to $4; blacksmiths, $4; teams, including driver, $5; machinists, eighty cents to $1 per hour. The work was done in the midst of the "boom" in Southern California, when labor of all kinds was difficult to obtain, independent and restless on account of the general feverish excitement, and inclined to pick up at any moment and move on in search of better pay. All classes of supplies, tools and materials were correspondingly higher than the ordinary prices. These conditions increased the cost of the work twenty to twenty-five per cent. above the normal.

There has been no lack of wiseacres who predicted the failure of the enterprise as a permanent irrigation scheme; and some of the most intelligent citizens of the country criticised the location of the reservoir so near the mouth of the stream, on account of its presumed liability to obliteration as a reservoir by reason of the deposit of sand and silt. A careful examination of the water of the stream at flood time, when it was most heavily charged with sediment, convinced the writer that fears of this nature were groundless. An estimate made, within reasonable limits, indicated one thousand years as the time when it might be expected to fill. Samples taken by the State Engineer Department of California of the water of the Yuba, Bear, and American Rivers, immediately below the hydraulic mines, yielded an average of only about one half of one per cent. of sediment. Were the Sweetwater as heavily charged, it might fill the reservoir basin in one to two hundred years, but the voids would still contain a considerable volume of water that would drain out and be available, and the utility of the reservoir would not be destroyed if it were entirely filled with sand.

The Distributing System.—From the dam to the lower end of the cañon, 1 600 feet, the main pipe is 36 inches in diameter, and covered with masonry laid in lime mortar, plastered with cement. From this point it is reduced to 30 inches diameter, and follows the valley for 5 miles, and thence rises to the top of the Chula Vista Mesa 92 feet above sea level. Its entire length is 29 800 feet, and at its terminus the water

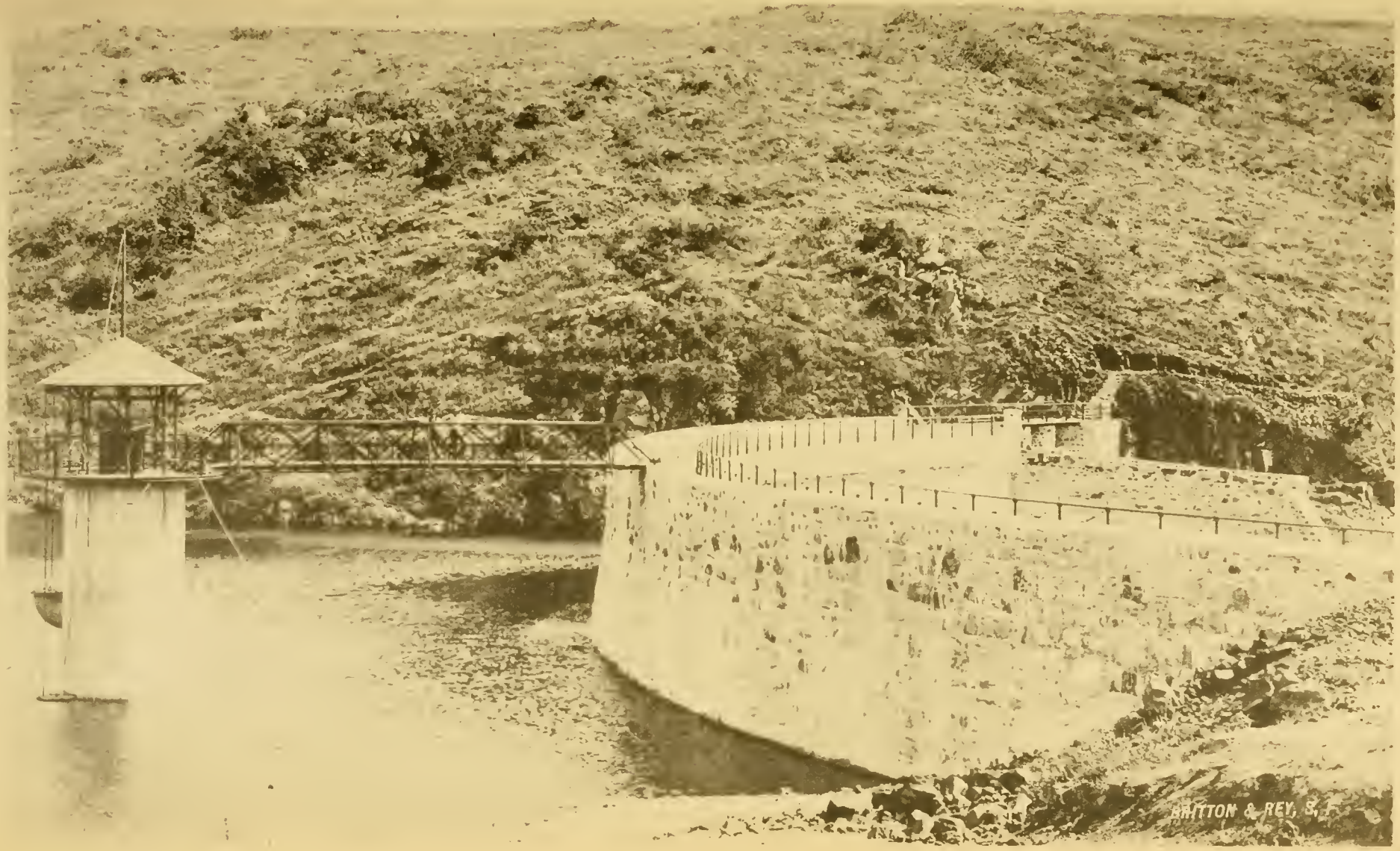


Plate XXVII.—Sweetwater Dam.

is divided into two 24-inch pipes, one running south 1 mile, the other west half a mile, where it is reduced to 18 inches diameter, and is carried northward to and through National City.

At the terminus of the 36-inch main a blow-off gate is located, to be used as a relief to the wasteway of the dam in case of a sudden flood which might exceed the capacity of the wasteway, or to draw off the water from the reservoir if, for any cause, it was desired to do so.

Wrought-iron pipes were used throughout. The total length of mains and laterals that have been laid is 58 miles, with 5½ miles on hand to be laid this season. They are of three classes, viz., straight double riveted pipe, manufactured and laid by the Risdon Iron Works, San Francisco; converse lock joint, kalamined lap-welded tube, made by the National Tube Works of McKeesport, Pa.; and spiral riveted pipe made by the Abendroth Root Manufacturing Company, New York. About 16 per cent. of the pipe was of the first class, 72 per cent. of the second, and 12 per cent. of the third. The length and diameter of each class furnished was as follows :

Risdon Iron Works, San Francisco.

Wrought-iron, straight riveted.....	36 inches diameter..	1 594	feet.
	30 " ..	28 213	"
	24 " ..	2 034	"
	18 " ..	16 468	"

National Tube Works.

Kalamined tube.....	12 inches diameter..	25 903	feet.
	8 " ..	7 620	"
	6 " ..	132 333	"
	4 " ..	50 745	"

Abendroth, Root & Company.

Spiral steel and iron.....	24 inches diameter..	5 950	feet.
	12 " ..	10 029	"
	8 " ..	4 020	"
	6 " ..	17 870	"
Total.....	.....	302 779	"

The introduction of spiral pipe into the system was unfortunate, as it does not stand the test of transportation across the continent, and will

have to be taken up and specially treated to make it water tight. It will answer very well for sub-irrigation, if it could be properly controlled, but as it is laid in streets and avenues that system is not desirable or conducive to comfort in traveling.

The total cost of the pipe lines was as follows:

Pipe	$301 928 80
Freight	39 183 03
Distribution	6 271 06
Gates	1 849 62
Materials, tools, etc	5 932 57
Right of way and miscellaneous expenses	2 968 00
Pipe laying	144 630 78
Total	$502 763 86

Probable Duty of the Works.—One of the most interesting questions to the Stockholders of the Company is the result that may be reasonably expected in the way of irrigation from such a reservoir. The assumption is made that in average years, say three out of five, the watershed will yield a sufficient supply to fill the reservoir, besides maintaining the consumption through the rainy season, thus starting on the irrigation season about May 1st with a full reservoir. From May 1st to October 1st is the average season of irrigation—about one hundred and fifty days. Where pipe distribution is in use, a fair average allowance in Southern California is a duty of ten acres per miner's inch (five hundred acres per cubic foot per second). There are instances of a much higher duty having been attained—a duty of even forty acres per miners' inch having been accomplished in one place. Allotting 700 000 000 gallons for the annual consumption of National City, and for loss by evaporation during the summer months, the remainder would yield a flow of 2 000 miners' inches per day for two hundred days; with a duty of ten acres per inch, this amount would irrigate 20 000 acres. In the course of time it is expected that a duty as high as twenty acres per inch will be reached, in which event, a reservoir full may be extended over two years' time, and still irrigate 20 000 acres, and afford a domestic supply to the town of National City.

Water rights, giving to the purchaser simply the privilege of becoming a customer for water have been sold on the San Diego Flume Company flume at the rate of $2 000 per miners' inch. At this rate the value

of the irrigation supply of the reservoir is $4 000 000. The construction of the works has already added a value of $1 500 000 to the principal tract of five thousand acres which has been supplied with a complete system of water pipes, and another million to the value of town property in National City, and lands in its immediate vicinity.

The Plates accompanying this paper are:

Plate XXIII. Front View of Sweetwater Dam; Mount San Miguel in the distance.

" XXIV. Rear View of the Dam.

" XXV. View from lower end of Wasteway looking Northeast.

" XXVI. The Waste Weir.

" XXVII. The Tower, showing one of the Inlets with Basket Screen in position.

" XXVIII. Plan of Sweetwater Dam.

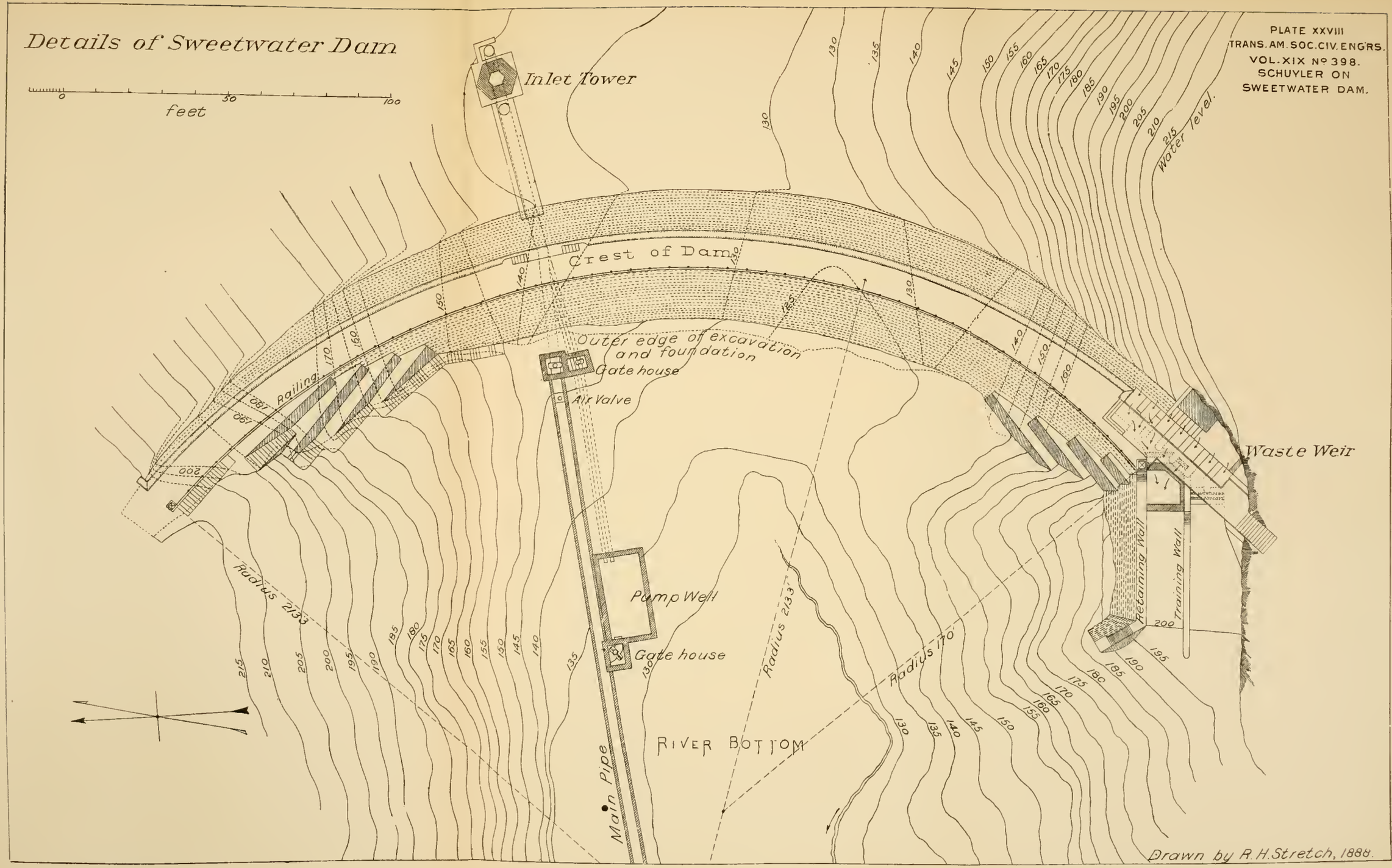
" XXIX. Elevation of Dam with details of Waste Weir.

" XXX. Section of Dam and Tower, with Outlet Conduit.

" XXXI. Map of San Diego Bay Region, showing location of the Sweetwater Dam and Reservoir and the District commanded by Pipe Lines from it.

" XXXII. View of Bear Valley Dam.

" XXXIII. Profile of Bear Valley Dam.


Details of Sweetwater Dam
0
30
100
feet
PLATE XXVIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XIX Nº 398.
SCHUYLER ON
SWEETWATER DAM.
Inlet Tower
Crest of Dam
Outer edge of excavation and foundation
Gate house
Air Valve
Railing
Waste Weir
Pump Well
Gate house
Retaining Wall
Training Wall
Radius 213.3'
Radius 170'
Radius 213.3'
RIVER BOTTOM
Main Pipe
Water level.
Drawn by R. H. Stretch, 1888.

Details of Sweetwater Dam

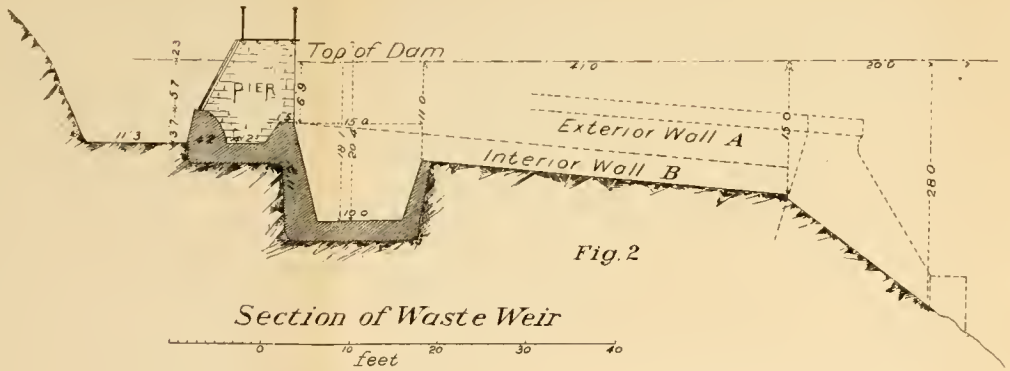


Fig. 2

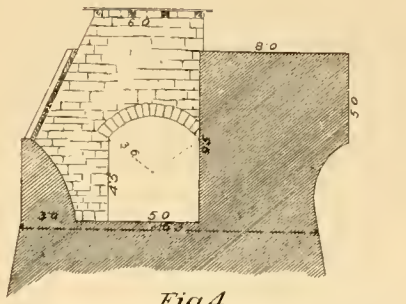


Fig 4
Section of Waste Weir
0 10 20
feet

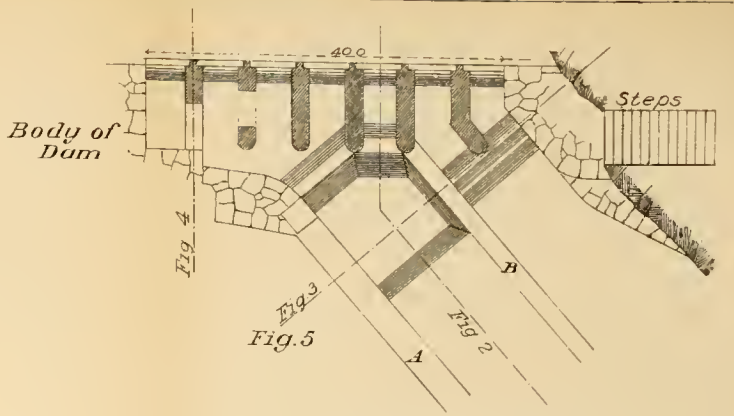


Fig. 5
Plan of Waste Weir
to locate sections.

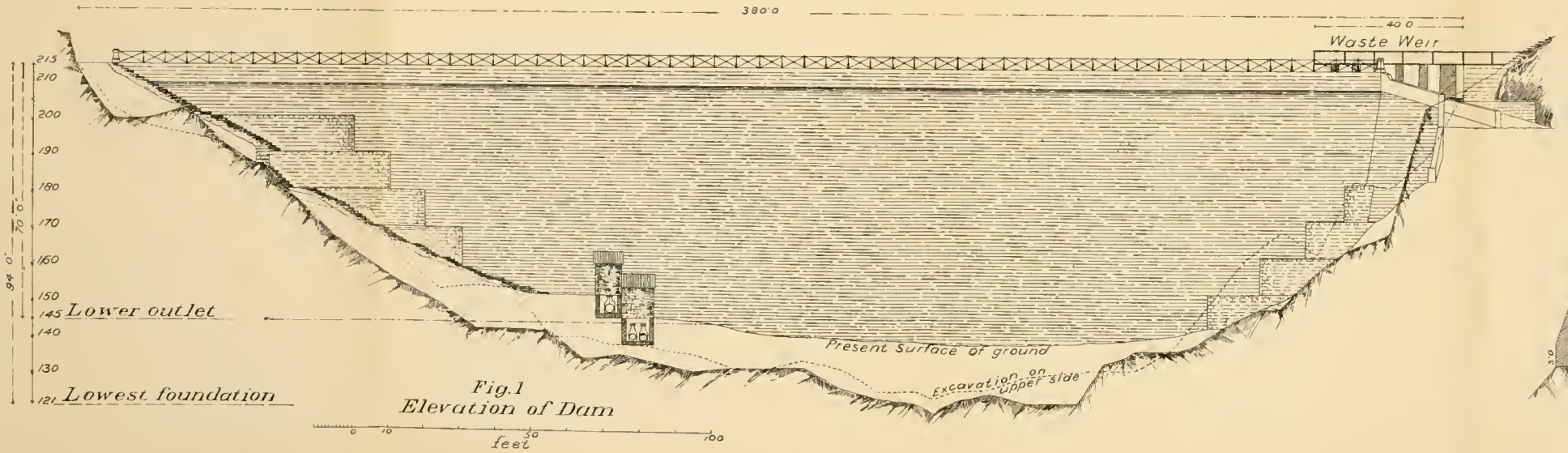


Fig. 1
Elevation of Dam

PLATE XXIX
TRANS. AM. SOC. CIV. ENGRS.
VOL. XIX Nº 398.
SCHUYLER ON
SWEETWATER DAM

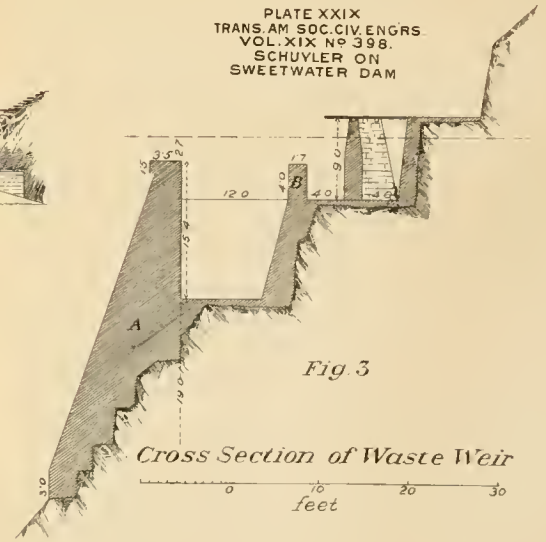
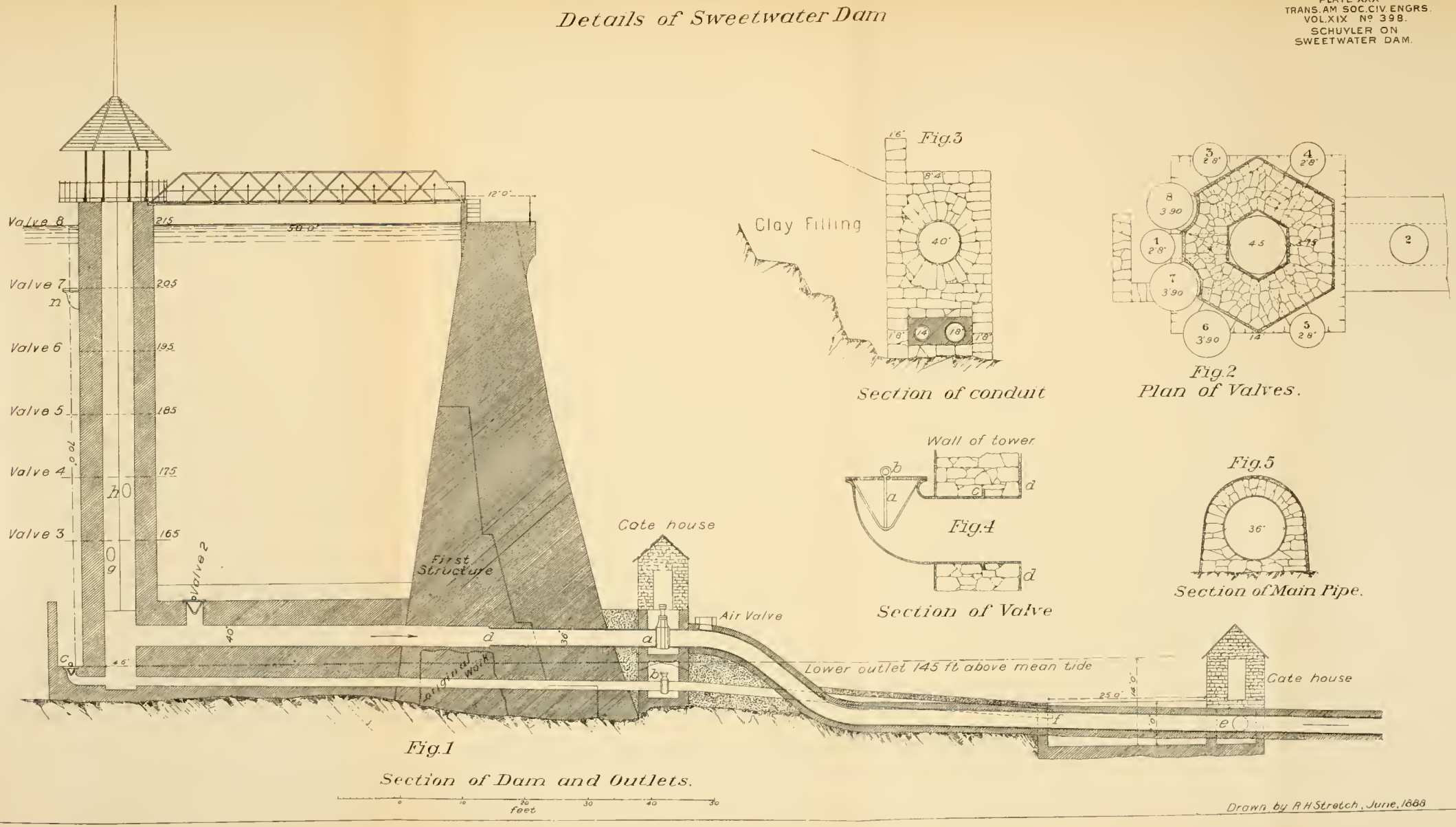


Fig. 3
Cross Section of Waste Weir

Drawn by R.H. Stretch. June 1888.

Details of Sweetwater Dam

PLATE XXX
TRANS. AM SOC. CIV ENGRS.
VOL. XIX Nº 398.
SCHUYLER ON
SWEETWATER DAM.



Drawn by R H Stretch, June, 1888

MAP OF THE

SAN DIEGO BAY REGION

showing the location of the

SWEETWATER DAM & RESERVOIR

and the District commanded by

Pipe Lines.

SCALE OF MILES 1 2 3 4 5 6

MAY. 1888.

PLATE XXXI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XIX Nº 398.
SCHUYLER ON
SWEETWATER DAM.

Drawn by R.H. Stretch, June 1889

DISCUSSION.

JAMES B. FRANCIS, Past President Am. Soc. C. E.—I do not think I understand enough about this particular design to criticise it much. One thing strikes me as dangerous, and that is the wasteway being stopped up by planks or flash-boards and the relying on any one's opening it on proper occasion. Floods come very suddenly sometimes here, and I presume they do in that region. There would be great risk in this method.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—What is the surface area of the reservoir when full?

The SECRETARY.—721.86 acres.

Mr. FRANCIS.—What is the area of the water-shed?

The SECRETARY.—The water-shed, tributary to the dam, 186 square miles, of which one-third is above an elevation of 3 000 feet.

Mr. FRANCIS.—The area of the reservoir is then the one hundred and sixty-fifth part of the area of the water-shed, and a depth of 1 inch flowing off of the water-shed, corresponds to a depth on the reservoir of 165 inches. I take it, in that region, a rain-fall of 6 inches in twenty-four hours, or even more, should be provided for, which would require a very large spill-way.

The SECRETARY.—The paper refers to occasional rainfalls of great severity.

Mr. FRANCIS.—It seems that extraordinary provisions should be made for the overflow.

JAMES D. SCHUYLER, M. Am. Soc. C. E.—The dimensions of the wasteway were proportioned to carry a volume equal to or greater than the maximum flood discharge of the stream, as it was indicated by high water marks left by the freshet of 1884, the highest in the recollection of the oldest residents. With a blow-off gate equal to twenty per cent. of the capacity of the wasteway, which may be opened on the approach of a flood wave in advance of the rise of the water to the floor of the wasteway, the gate-keeper has full control of the floods. Moreover, before the parapet wall could be surmounted, the capacity of the wasteway would be double the flood discharge of the freshet of 1884, or about 3 000 cubic feet per second.

The flash-boards are not intended to be used until the end of the rainy season, to accumulate the later flow of the stream before it dwindles to its summer stage.

E. WEGMANN, Jr., M. Am. Soc. C. E.—In discussing the excellent paper which has just been read, the first point to which I wish to call attention, is that in this dam, as in all similar structures built recently in California, the plan of the wall is curved, being convex up-stream. While all engineers will readily admit that the curving of the plan adds

additional strength to a dam which has been built across a narrow valley, considerable difference of opinion exists as regards the advisability of adopting this plan of construction in the case of a wide valley. The pressures in the masonry resulting from a curved dam, acting as a horizontal arch, depend upon the radius of curvature and upon the thickness of the dam.

In considering the strains in the masonry, which may result from a dam acting as a horizontal arch, three cases may occur:

First.—The dam may act perfectly as an arch.

Second.—It may act imperfectly as an arch.

Third.—It may not act as an arch at all.

In the first case, the pressures in the masonry are to be calculated by the usual formulas.

In the second case, in which the amount of pressure transmitted by the dam to the lateral sides of the valley is supposed to be uncertain, we ought, in order to insure safety, to take an extreme assumption, namely, that the whole thrust of the water might have to be borne by the dam acting as an arch, which would make this case the same as the first.

Finally, the dam may not act as an arch at all, in which case the curving of the plan by adding to the length of the wall, would involve a waste of masonry.

That the stability of a dam may depend upon its acting as a horizontal arch, is proved by the Zola Dam in France, in which the line of pressure, reservoir full, falls considerably outside of the base of the wall. But in this case, the valley is narrow and the radius of curvature small, so that the maxima pressures arising from the arch action in the dam are within the limits of safety. Should we have, however, to deal with a wide valley, requiring a radius of about 1 000 feet for curving the plan of the dam, the pressures resulting from the arch action, as calculated by the usual formulas, would be found to be excessive. In all probability no part of the water-pressure is transmitted by a dam to the sides of a wide valley, as, owing to the length of the masonry, a considerable compression would have to occur before even a moderate amount of pressure could be transmitted.

Just where the distinction between a wide and a narrow valley should be made, must remain, in our present state of knowledge, a matter of judgment. A good rule to follow is, not to curve the plan of a dam, when the pressures arising in its masonry from its acting as a perfect arch are in excess of what is considered safe.

W. Barclay Parsons, M. Am. Soc. C. E.—What does Mr. Wegmann consider the limit of radius?

Mr. Wegmann.—I cannot give the exact radius, but a pressure of 14 tons per square foot should not be exceeded.

Mr. Parsons.—Acting as an arch?

Mr. WEGMANN.—Yes, sir; taking into consideration the dam acting as an arch.

Mr. PARSONS.—What radius is that?

Mr. WEGMANN.—I should think that it may be five or six hundred feet. The Zola Dam depends entirely upon its arched form for stability; it would be overturned were it not for its curved plan. This is the only case of this kind I recollect, unless the Bear Valley Dam, built in California recently, is another example. In this dam I think the line of pressure is very near the toe. The Bear Valley is narrow, and I think in that case there is no doubt but that the dam should be arched.

J. J. R. CROES, M. Am. Soc. C. E.—The pressure in pounds per unit of surface depends on the thickness of the dam at the elevation under consideration. The total arch pressure exerted at any point is equal to P times R; that is, the pressure per square foot on the face multiplied by the radius; this represents the total horizontal thrust on a voussoir joint, and this must be divided by the thickness of the dam in feet, to give the arch pressure per square foot of the joint.

Mr. SCHUYLER.—The Bear Valley Dam in San Bernardino County, Cal., cited by Mr. Wegmann, has no parallel among masonry dams in the civilized world. Its dimensions are so bold, it is so wide a departure from all previous theory and practice, and it has so long stood against the pressure of a full reservoir, that its profile is worthy of reproduction. See Plates XXXII and XXXIII.

At base it has a thickness of 22 feet, and it was carried up to the height of 16 feet the first year, and left in that condition during the winter. In the spring following, finding the cost of the original plan was to be greater than its projectors could afford, the structure was narrowed to a thickness of 8.5 feet, and carried to a height of 46 feet further, or 62 feet in all. At the top its thickness is but 2.5 feet.

These dimensions are more slender than have been heretofore reported, but I have taken pains to verify their correctness.

Considering the upper 46 feet of the dam independently, the resultant of gravity and water pressure (reservoir full) falls fully 15 feet outside the toe, or nearly twice the thickness of the masonry. Its radius is something over 250 feet, and the arch thrust on voussoirs is over 40 tons per square foot. The material is granite, laid in cut blocks, with Portland cement. The stability of the structure is, of course, entirely dependent on its arched form. The whole of the upper 46 feet acts as an arch, and withstands strains far beyond the theoretical limit. That it has stood for so long has been a matter of surprise to engineers who have known of its extraordinary frailness, but the fact that it has stood, with a full reservoir behind it (an enormous reservoir, too, covering some 2 000 acres, and impounding 10 000 million gallons), for at least three

years, should convey some useful suggestions on arch action in masonry dams worthy of careful study and investigation by the profession. It would seem to be a standing menace to the thousands of acres of vineyard and orange groves that lie in the valley below, and yet it remains a standing marvel, as curious as it is instructive. The company owning the dam are proposing to strengthen and increase the height of the structure.

Mr. WEGMANN.—Professor Rankine states that all calculations treating a dam as a horizontal arch are so indeterminate that they are of no value. He recommends, therefore, that a dam should be made sufficiently strong to resist by gravity alone, and that the plan be curved as an additional safeguard whenever the local conditions make this advisable. Of course, that leaves still a wide field for judgment. I should limit the curved plan to valleys of about 800 feet or less.

MR. CROES.—There are cases, where, in a valley of over 500 or 800 feet in width, a straight line would not be the most economical.

Mr. WEGMANN.—That would depend upon local conditions. The question of architectural effect may also deserve consideration; a curved dam may give a finer effect. The curved plan can do no harm under any circumstances, because when the valley is very wide the dam will simply not act as an arch; the only damage being a waste of masonry.

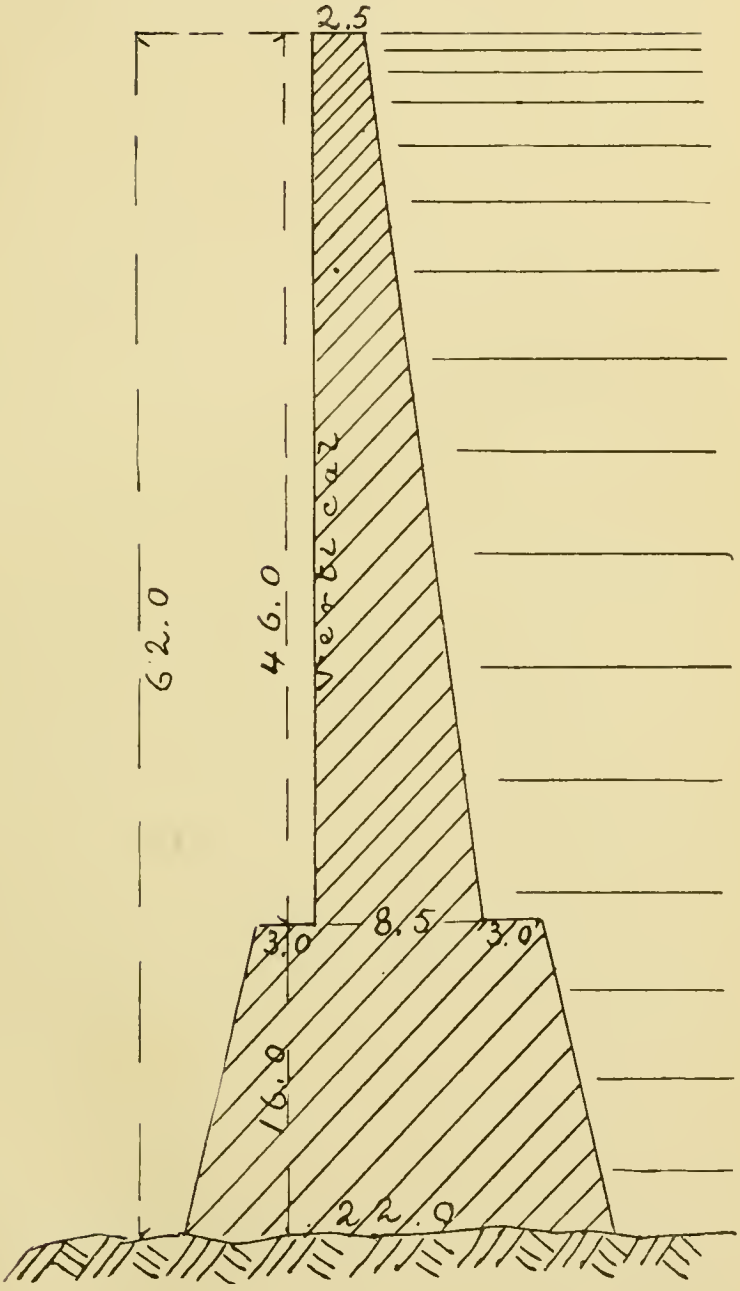
L. L. BUCK, M. Am. Soc. C. E.—Some curiosity is expressed to know why, if it was necessary to curve this dam at all, two different radii were adopted. The pressure against all points of the dam at equal depths below the surface of the water is equal, and hence it should ordinarily have a uniform curvature, if any, for the reason that, by compounding the curve, the flat portions will tend to thrust the sharp portions up-stream. I do not intend by my question to criticise the strength of the dam, but to ascertain the motives for compounding the curve.

Mr. SCHUYLER.—An examination of the plan will reveal the fact that on the up-stream face the curve is carried around with but one radius. It is only on the lower face next the waste-way that the compound curve was introduced to increase the thickness of the wall and relieve any possible shock that might be sustained by an extraordinary flow of water over the waste-way, as well as facilitate an increase of width of the latter. The compounding of the curve for a short distance simply adds a little thickness at one end of the dam in a curved wedge form, and does not affect the structure in any way except to increase its stability.

Mr. BUCK.—It may not be out of place here to say a few words regarding the subject of straight *versus* curved dams.

The dam is a retaining wall designed to hold the water of a river or other stream, to resist the pressure of this water and transmit such

Plate XXXII.—Bear Valley Dam.



PROFILE OF BEAR VALLEY DAM.—Plate XXXIII.

pressure to such immovable, natural foundation as the site selected will afford. It should be the aim of the engineer to effect this result with the least expenditure of capital that will render the structure perfectly safe. The nature of its service is such that it must retain its integrity of form, connection of parts to each other, and to the bottom and sides of the ravine or valley in which it is located.

Of the forms of masonry dams, there are two which have received the most attention in the discussions upon dams, viz.:

First.—A straight dam, with sufficient thickness to enable it, by its weight and strength, to safely resist the pressure brought against it by transmitting such pressure directly down to the natural rock on which the dam is built. In such a dam, the masonry on the down-stream side is compressed in excess of what pressure its own weight produces, while that produced on the up-stream side, caused by its own weight, is reduced by the pressure of the water. The effect of this action is to depress the lower side and elevate the upper. Consequently, the dam must lean down-stream to a greater or less extent, dependent upon the ratio of its height to its thickness. If the dam is properly designed and constructed, the amount of such leaning will be very slight. This is the most simple form of masonry dam.

Second.—This form of dam is curved in plan so as to present its convex side up-stream. The intention of this curved form is to cause the structure to act as an arch and transmit the pressure of the water to the sides of the valley or ravine. As the materials of which this arch is composed are shortened by such pressure, which means the shortening of the arc of the dam, while the ends must not approach each other, the curve will be flattened. Consequently this dam will also lean down stream. The amount of such leaning will increase in some ratio with the length and height of the dam, and the decrease of its curvature and thickness. It must have thickness enough to do its duty as an arch, to prevent the water finding its way through or underneath, and as the bottom must not move, it must be thick enough and heavy enough to prevent any part sliding on that below; or the whole sliding on the foundation. But if the thickness is too great, it may not allow the dam to lean enough to get the benefit of the arch form without cracking the masonry on the side exposed to the water. In this form of dam there can be no arch action at the bottom, nor, practically, for a considerable distance above, while its maximum is at the top. Hence it is practically a combination of the two systems and subject to the complicated stresses always attending such compound structures. As it must not have too flat a curvature, and as the chord of its arc must be somewhat less than twice its radius, it could, at best, be adapted only to situations where the valley is narrow in proportion to the height of the dam. In fact, its application should be limited to cases where the site is a very narrow, deep rock cañon or fissure, in which the distance from the middle of the

top of the dam to the solid rock at the sides of the cañon is not greatly in excess of that from the top to the solid rock at the bottom. Hence, remembering, 1st, that it is masonry that we are considering; 2d, that in the simplest structures of masonry, where durability is required, engineers consider it necessary to adopt a high factor of safety; and 3d, that the masonry structure is a dam requiring the most perfect integrity of construction in every part, it must be admitted that, when constructed on such a compound system as this last, involving so many uncertainties, it has no claims to being considered a scientific structure.

The idea of combining the two forms of dam, considered above, has gained some adherents among engineers. As I understand it, the plan proposed by its advocates consists of a curved dam. It is proposed, at the same time, to make it in every respect strong enough to be perfectly secure, had it not have been curved, but it is curved as a means of additional precaution. Here then we have a structure primarily built upon a system in which the pressure is transmitted to the solid rock on which it rests, possessing abundant stability, having a form of cross-section which cannot, and is not expected to, change (excepting to an almost inappreciable degree), and yet which it is proposed to render additionally secure by giving it a plan that could only accomplish its object by transmitting the pressure to very much more distant points, and could only do this after the first system had failed. No claim is made that the rock at the bottom does not afford every bit as reliable a foundation as that at the sides, or that it is not in a perfectly legitimate direction to transmit the pressure. And yet the advocates of this plan propose to increase the cost of the structure, by ten or fifteen per cent., by introducing such a complication as this. They find no fault with the simple and thoroughly scientific system of the gravity dam, in which the strains all lie in parallel direction, and end in the best possible foundation. Will some of them explain why, if additional security is required, it would not be best to seek it by simply extending the system on which the structure is designed?

Mr. Wegmann.—The profile of the Sweetwater Dam has evidently been designed on scientific principles, and differs probably but slightly from a theoretical type having the minimum area. Among the many masonry dams which have been built within recent years in strict conformity to the requirements of theory, only one—the Habra Dam—has failed, and that was on account of bad workmanship and poor materials. The maxima pressures in the masonry, which in the first scientific profiles was limited to 4 to 6 tons per square foot, reached in recent structures 8 tons per square foot. In these dams the extreme positions of the lines of pressure, reservoir full or empty, are generally made to lie within the center third of the profile. I presume that this condition has been observed in the Sweetwater Dam.

The CHAIR.—I think Mr. Schuyler said they were inside the middle-third.

Mr. WEGMANN.—It would be interesting to ascertain that fact.

The SECRETARY (read from paper).—"In designing the plan of the higher structure," etc. (See page 206.)

Mr. WEGMANN.—The design seems to be bolder than I thought. As the line of pressure, reservoir full, falls considerably outside of the center-third of the profile, I think it would be hazardous to trust to the dam's strength to resist overturning merely by its weight. As the valley is narrow, however, at the site of the dam, the curved plan adds undoubtedly to the strength of the structure.

The weight of the masonry is given as 164 pounds per cubic foot. I know of only one dam in which the masonry was equally heavy, viz., Vyrnwy Dam near Liverpool. The stones of which the Sweetwater Dam was built must have had a very high specific gravity.

The SECRETARY.—The weight is given as 175 to 200 pounds per cubic foot.

Mr. WEGMANN.—That accounts for the great weight of the masonry.

The SECRETARY.—The stones placed in the wall are also said to have been large.

A. FTELEY, M. Am. Soc. C. E.—In connection with the question of weight of masonry, I will refer to two experiments made in 1885, to ascertain the probable weight of rubble stone masonry, such as might be used for the proposed construction of Quaker Bridge Dam.

Two blocks, about 1 cubic yard in volume, were built on a platform of rough stones, ranging in size from a common spall to about 3 cubic feet. The mortar used was made of Portland cement, mixed in the proportion of one of cement to two of sand. The stones used appeared to be gneiss.

After several months, the two blocks were measured and weighed with care, and found to have an almost identical weight of 156½ pounds per cubic foot. The computations subsequently made to determine the profile of the dam, were based on the weight thus ascertained.

Mr. CROES.—Would it be advisable to assume that a large mass of masonry would have the same specific gravity as small samples prepared in the manner indicated?

Mr. FTELEY.—The two samples just mentioned were taken from two different quarries in the vicinity of the proposed structure. The experimental blocks were built in the same manner as the mass of the dam should be built, and although it cannot be stated that the whole structure would be the same throughout, it seems fair (especially when it is admitted that the stones used in the final structure will be generally of a large bulk) to state that it is safe to use in the computations the weight determined experimentally.

Mr. Francis.—One point occurs to me in connection with a curved dam. Suppose one to be built depending entirely on the curved form or arch; when the pressure of the water comes against it there would be a compression of the material and the arch would be flattened, which would cause it either to crack or slide on the bottom. I am taking an extreme case, that of a dam depending entirely on the arch form.

Mr. Fteley.—Mr. Rankine, when speaking of the advisability of adopting a curved form for the plan of a masonry dam, says that "in the present state of science, the calculations of stability, treating the dam as a horizontal arch, are so uncertain as to be of doubtful utility." If, in connection with the above we consider a very long dam, the length of which is several times greater than the longest known arch, and built of rough stones without any attempt at any radial disposition of the joints, the superiority of the arch form does not appear a matter of certainty.

In such a case it is much more reasonable to suppose that whatever be the plan of the dam; straight, curved up-stream or down-stream, or with any irregular shape that may be determined by the topography of the ground, a failure would occur as it would in a retaining wall of indefinite length, between two weak points.

The uncertainty of relying upon a long curved dam as an arch, is so well recognized by those who advocate the curved form, that they recommend at the same time, as the most essential part of the design, a gravity section with a large margin of safety. Moreover, they admit that, in a curved dam of large span the structure will not act as an arch except after the force of gravity has been overcome. In such a contingency the condition of the structure designed to resist by its own gravity (say) twice the greatest expected strain, may well be imagined, and its chances to be then held by arch action would appear decidedly uncertain to those living below.

If the arch form, or any other form, is more economical than the straight line, it would be well to consider it favorably; if not, it is not advisable. Should it be advocated on account of architectural beauty, it remains with those in charge to determine whether the additional expenditure would be justified by the superior appearance of the structure.

C. B. Comstock, M. Am. Soc. C. E.—Of course the strains at the bottom of the dam would be carried by the rock on which the dam rests, and as the lower course of the dam cannot move on this rock the arch effect there could not come into play at all; the arch effect only comes into play when there is some arch compression. When you strike the center of a vertical arch the crown sinks, the arch changing form. That cannot take place at the base of the dam. But at the top of the dam, although the water pressure is less than at the base, this pressure

acting as on the top of a vertical beam fixed at its base, bends the top of the dam down-stream. This bending compresses the horizontal arch formed by the top of the dam, and transmitted to the abutment, increases in some degree the power of the dam to resist overturn, considered only with a gravity section. You get some additional strength by giving it the curved form.

A gentleman suggests that there would be cracks in the dam. Of course, if the strains on the dam were nearly enough to break it by overturning there would be cracks on the upper face. I am supposing a dam so strong that those cracks would not take place, the bending of the dam not being sufficient to produce them. Then the arch resistance would increase the strength of the dam and reduce the factor of safety needed for a gravity section.

Mr. Croes. — Some experiments have recently been made by a board appointed by the Government, on the strength of brick arches, and the conclusion reached by the board was that these brick arches did not act as arches but as beams; that the masonry was monolithic, and that the strains were not arch strains at all, but were simply beam strains.

Mr. H. W. Brinckerhoff said that these experiments were referred to in the *Engineering and Building Record* of October 13th, 1888, and mention was there made of experiments noted in Vol. VIII., Van Nostrand's Engineering Magazine, October, 1872, as to the action as beams, of arches of brick.

Theodore Cooper, M. Am. Soc. C. E.—I do not agree with the idea advanced that a masonry dam of large radius can exert much resistance as an arch. To do so it would be necessary for the arch to slide upon the foundations or at some intermediate points—an idea which no one would consider as possible or desirable. For a curved dam to act in any manner as an arch it must move from its original position. The bottom being fixed, motion can only take place by the bending of the cross-section down-stream, which means that the greatest arch action will be at the top and nothing at the bottom. This bending down-stream of each cross-section induces also crushing strains at the toe.

So that we must have, with our arch action, a resistance from the cross-section acting similarly to a pure gravity form. What is the relative value of the two actions ? Arch action is dependent upon the amount of the deflection of the top of the dam down-stream. The crushing effect of the toe is measured by the same deflection. By assuming or calculating this deflection for a dam having a cross-section like the one before us, I am confident that the amount of arch action which could be found for a safe toe pressure, would be relatively very small, especially for long radii. The calculation is not a difficult one, and the least known

element of the calculation, the modulus of elasticity, entering into the deflection for both cases, can readily be eliminated.

After finding the theoretical amount of the arch resistance, we would still hesitate very much in assuming it to obtain to any such extent in actual practice owing to the absence of voussoirs and of homogeneity.

General Comstock.—I did not propose to estimate the amount; I merely stated that I thought there was some resistance occurring in that way. I suppose if you were to take the top of that dam and press it with your hand you could bend the dam down-stream in some degree, greater or less, and the moment you bend the dam down-stream you bring into play the pressure of the arch, which adds a little to the strength of the dam against overturning. The moment you bend the dam down-stream you transfer a little pressure to the ends of the arch.

Mr. Cooper.—We agree technically, but where we differ is in the amount that would be gained. I think there would be a very small amount indeed. The dam is so large at the base in proportion to its height, that the deflection would be very minute.

General Comstock.—The section of a dam 90 feet high, as I believe this was, would bend in some degree. The stone arch of the Chester Bridge of 200 feet span sank $2\frac{1}{2}$ inches at its crown at decentering, while for the Gloucester Bridge the sinking was 1 inch. As these sinkings gave severe compressions to the arch, it is probable that a sinking of $\frac{1}{10}$ of an inch would give a considerable compression in an arch not very flat.

For the purpose of estimate, suppose a foot in length of the dam to be replaced by a foot of granite in a single block 15 feet wide and 90 feet high; owing to its homogeneousness this block would have a much higher modulus than the actual dam, which, though thinner at top, is much thicker at its base. A horizontal down-stream pressure of a ton at the top of this stone would bend it down-stream by about $\frac{1}{30}$ of an inch, if W. Thomson's value of Young's modulus for stone be used, namely, 350×10^6 grammes per square centimeter. But if the value given in Hütte, 1883, be used (which seems much too small), a much larger deflection would result. As a ton's pressure would result from the top 9 feet of water, it seems possible that such a dam under the whole pressure might bend down-stream by $\frac{1}{10}$ of an inch.

Mr. Schuyler.—This estimate by General Comstock of the probable amount of bending of the dam down-stream with strains arising from a full reservoir, is quite reassuring. A movement of $\frac{1}{10}$ inch, or even twenty times that amount, might take place without appreciable results in the way of rupturing the masonry.

It seems clear to my mind that gravity and arch action must go hand in hand. When the reservoir is partly filled, in the case of the slender-

est of the pure arch type of dam, there is a point where the arch is not called into action, and resistance to pressure comes solely from the gravity of the structure. As the water rises the gravity point is passed, and then the excess of strain beyond the amount which the gravity of the mass is capable of resisting, is transmitted to the abutments. I do not think it can be maintained that when arch action begins, all the strains produced by water-pressure are necessarily borne by the arch, or any part of them, except those which the gravity of the dam cannot withstand. The dam must resist strain by its gravity to the extent of its weight, be that more or less, and from that point on it calls in the arch to its aid, and the two act together. You cannot get rid of the gravity of the mass in an arched dam. It is always there and always acting against an overturning force to the extent of its ability. I cannot conceive by what reasoning it should occur to any one that in order to permit a dam to act as an arch it should be necessary to increase the profile. If the profile is designed of such dimensions that it will resist by gravity the quiet pressure of water, the arch is not called into action until the pressure is increased by some extraordinary event, such as wave action, ice-thrust or earthquakes. This is the basis of reasoning upon which the profile of the Sweetwater Dam was designed. It was not designed to rely wholly upon the arch for its stability. If it had been, the experience of the Lola and Bear Valley dams would perhaps have justified a more slender profile. Nor was it intended to rely solely upon gravity for its factor of safety. It is stable under quiet pressure, and its arched form is believed to protect it from extraordinary shocks. The effect it was to produce upon the public mind in the inspiration of confidence was not disregarded, and in this respect its arched form adds such an appearance of solidity that it has been a marked success. It has been visited by thousands from all parts of the world, and no doubts are ever expressed of its thorough stability. It inspires the admiration, respect and confidence which every dam should inspire.

Mr. Cooper.—It has struck me, that the point which the author did not explain, is the building of three dams, one on top of the other; I do not think the author has anywhere given a description of the manner in which he put those three structures together.

Mr. Croes.—I think there is a deficiency in the paper in that respect. There was a large dam built before this was begun, and no description is given of the manner in which the masonry of the old dam is joined to the new one. If they were built in the same way, and care was taken, to make the joints between the old and the new work good, and the new work had time to set thoroughly before pressure was put upon it, I don't suppose it would make much difference.

Mr. Wegmann.—The dam was remarkably water-tight; is not that a proof that the joining must have been made thoroughly?

Mr. Croes.—The writer states that the dam is perfectly water-tight, but judging from some of the photographs of the dam here there is very evidently water coming through the dam at the point at which that joining was made.

The Secretary.—It is suggested that that may be water or it may be mortar.

Mr. Croes.—The mortar would not run out in that way unless there was water to bring it out.

A Member.—The author did not say that the dam did not leak ; the conduit did not leak.

Mr. Croes.—A few months ago I visited the Boyd's Corners Dam in Putnam County, N. Y., which is 60 feet high and was built about eighteen years ago, and there were in places streams of water running down the face of it. Immediately after this dam was built an earth embankment was built against the up-stream or water face. The joints in the masonry on this face of the dam were never thoroughly pointed before the earthen embankment was put against it. About 30 feet below the flow line there are places where the water is running down the face of the dam between the stones and the joints appear to be full of mortar, except at these few places. The face of this dam is of stones, from 2½ to 5 feet deep (bed), and from 18 to 20 inches rise, and the heart is of concrete, yet the water gets through it.

There was a small dam built of rubble masonry at New Rochelle, N. Y., about two years ago, which leaked very badly when the reservoir was first filled, the water passing through the masonry and spouting out with considerable force from some of the joints on the lower face. The proprietors had not considered it necessary to have the work inspected by an engineer during its construction, but called on one to remedy the defects which appeared on its completion. Mr. C. W. Hunt, M. Am. Soc. C. E., had charge of the repairs.

Charles W. Hunt, M. Am. Soc. C. E.—On examining the water-face it was found that there were a good many joints not properly filled with mortar. The water was then drawn down gradually, and masons working from a raft raked out the joints in the face and filled them up with cement mortar, well rammed in. Holes were found in which a stick could be pushed in three or four feet without meeting with any obstacle. In some holes as much as eight or ten pailsful of mortar were rammed in. The whole water face was pointed up in this way with well compacted mortar, and the dam does not leak at all now.

Mr. Schuyler.—The original structure of the Sweetwater Dam was encased by the secondary dam on both sides, as shown by the cross-section. It was narrow, and low, irregular in form and surface, with

numerous re-entering angles ; and the union of the new masonry with the old was perfectly formed, with a grout of richer mortar than was commonly used. The bond was similarly formed between the secondary dam and its extension. The steps on the lower face of the former gave seat for the new work, and the omission of pointing mortar, as well as the irregularity of the face stones made the bond a good one. The work was allowed to set several months before it was subjected to much pressure.

As stated in the paper, the leakage through and under the secondary dam was entirely cut off by the new toe, but when the gates were closed and the reservoir allowed to fill, sweating took place through the masonry, principally above the level of the top of the secondary dam, and along the sides, through the minute seams of the bed-rock, carrying with it an efflorescence, forming an incrustation on the face which appears in the photographs. This sweating has dried up to a large extent. The total leakage through and under the wall at its maximum measured about 60 000 gallons daily. It is now reduced to about one-fifth that quantity, the greater portion of which is in minute trickling streams through crevices under the dam. All the leakage comes through without pressure, and can be stopped on the lower side, by pricking cotton or any fibrous substances into the tiny pinholes in the mortar through which it oozes. There is nothing like a jet or spurt of water anywhere apparent.

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SOME OBSERVATIONS ON THE SUBJECT OF THE IMPROVEMENT OF SEVERAL OF THE RIVERS OF THE ATLANTIC COAST.

By W. P. CRAIGHILL, M. Am. Soc. C. E.

PRESENTED FOR THE ANNUAL CONVENTION, JUNE 28TH, 1888.

WITH DISCUSSION.

It has been suggested that a short article, descriptive in a condensed way of the treatment of several of the tidal rivers of the Atlantic coast, with some comments thereon, may be interesting as a basis for discussion by the Society. What follows is the outcome of that suggestion.

It is proper to observe in the outset that brevity requires the avoidance of much detail. Should a discussion ensue, what it may lead to remains to be seen. The writer also wishes it to be understood that while the statements of this article are based upon his own experience, he desires to abstain from the didactic or dogmatic style, which is nowhere more unbecoming than in writing about the manipulation of the great forces developed by water moving in the channels of rivers, where

so many of the elements to be taken into account are constantly changing, and so much is uncertain because undiscoverable even after long study and observation. One ounce of experience is worth tons of mere theory.

The art of river engineering is much like that of the physician. There are great principles to be followed by the practitioner in each of these professions, but their application must be varied to suit the physical constitutions of the patients, and all these differ from others at the same time, and from themselves at different times. Underneath all the knowledge that study and experience can give must be a large stratum of hard common sense, or the result will be failure. The wise man is he who learns from his failures as well as from his successes. The physician has the advantage of the engineer that many of his failures are hid from mortal sight, whereas rivers badly treated not only live on but are sure to give full exhibition of the mistakes practiced upon them. Nature will avenge herself.

Improvement of the approach to the Harbor of Baltimore.

This improvement is partly in the Patapsco River below Baltimore, and partly in Chesapeake Bay, below the nominal mouth of the river. The Patapsco, below Baltimore, though called a river is rather an arm of Chesapeake Bay (see Plate XXXIV*). Above Hawkins' Point there has been always a greater depth than below. In early days the main obstruction to navigation was the great shoal extending southeastwardly below Hawkins' Point to the Seven-Foot Knoll. Through this shoal there was originally a meandering channel with a depth of about 18 feet at mean low water with an average rise of tide of 18 inches. Eastward of the Seven-Foot Knoll the channel had a somewhat greater depth, as shown on the map herewith, on which its original direction towards Swan Point and thence southward to the deep water of the bay is still given.

The first proposition to give a channel of a greater than the natural depth appeared about thirty-five years ago, and came from the late Colonel (then Captain) Henry Brewerton, of the United States Corps of Engineers. The plan was to give a straight channel along the line of the existing Brewerton Channel, with a depth of 22 feet at mean low water, and a width of 150 feet; which was to be excavated by dredging wherever necessary in the original line towards Swan Point. There

*It will be observed that the illustrative maps have been made as small as possible to avoid expense and to conform to the size of the volumes issued by the Society. Those who wish to see a map showing the relations of the Patapsco to its surroundings are advised to refer to some large atlas or specially to the printed map of the Coast Survey, No. 384, corrected to 1887.—W. P. C.

were few dredges in the United States in those days. After advertising for proposals for two or three years, no contractor could be induced to undertake the work at a reasonable price. Four dredges of the Osgood pattern were thereupon built by the United States, with dumping scows and an attendant tug-boat.

The work went on slowly, but as rapidly as the meagre and intermittent appropriations of Congress allowed, until brought to an abrupt termination by the war of 1861–65. With the revival after the war came a change of plan. The commercial interests of Baltimore demanded a greater depth of channel, which was increased to 24 feet at mean low water, with a width of 300 feet. With these changes came also another, viz., of location of the portion of the channel eastward of Seven Foot Knoll.

It had been observed that it was difficult if not impossible, except by constant dredging, to keep the channel east of Seven Foot Knoll deeper than nature made it. The cause of the filling was obvious—the sweep of the currents directly across it, both on the flood and the ebb of the tide, to say nothing of the fluvial matter brought down from the Susquehanna in freshets.

A careful study of the situation by the engineer then in charge called attention to two important conditions: The first was, that, considering as two great forces the two great masses of water meeting near the line from North Point to Bodkin Point, their resultant might be expected to be in direction nearer that of the greater, and in plan near the western rather than the eastern shore. With this idea in mind, study of existing maps indicated that, in an area not very distant from the western shore of the bay below Seven Foot Knoll and parallel to it, there was much deep water, with free passage through it, interrupted by lumps marked "hard." An examination of these lumps showed their hardness to be due very much to layers of oyster beds, living and dead, but with deep water quite near them.

It seemed reasonable to conclude that nature had been doing her best to make a deep channel in this natural position, and that, if these hard masses were removed, good results would follow. And so they did, the dredge being the agent. The change of plan was approved in Washington. Liberal appropriations were made by Congress, which, in order to expedite the work, were supplemented by the City of Baltimore under the wise lead of Mr. John W. Garrett and other public-spirited citizens. The channel was completed in 1874, the depth being 24 feet at mean low water, and the width 300 feet. To give this depth to the City of Baltimore, some dredging was also necessary above Hawkins' Point. The effect of the improvement in increasing the commerce of Baltimore was very marked, as also in changing the character of the vessels carrying it, as it permitted for the first time the use of the larger steamships suited to trans-oceanic navigation.

Another improvement in the channel was suggested: the excavation of what is now called the Cut-off, which is in the line of an old swash-channel. This was also a natural channel, but much of the material through which it ran was too hard to be removed by the water alone.

It had been observed that the lower portion of the shortened Brewerton Channel, that is, the part between the tangent to the western shore above North Point and the intersection of the Brewerton and Craighill Channels, was more liable to deterioration than the portion westward of that tangent, and it was thereupon determined to adopt as the main channel the line of the Cut-off, and abandon the portion of the Brewerton Channel below the upper end of the Cut-off. This idea has been carried out in connection with a still farther enlargement of the dimensions of the channel demanded by commerce. The plan now in process of execution is to provide a depth of 27 feet at mean low water, w th a width of 600 feet. The depth of 27 feet is already secured to Baltimore. A width of 400 feet with that depth exists throughout the Craighill division up to the Cut-off and through the Cut-off. Above the Cut-off to Baltimore the width is but 300 feet.

The advantages to Baltimore from the improved channel are obvious. Formerly, large ships could only come to the city or leave it with partial cargoes. Lighterage was done in the bay, some 20 miles below. This branch of business has disappeared altogether. When the Great Eastern came up Chesapeake Bay she was obliged to remain as far down as the vicinity of Annapolis. Now she could readily come to Baltimore.

The small vertical range of the tide, averaging 18 inches, has made it necessary to dig deep to obtain a sufficient depth for navigation at all stages of the tide. But this small vertical movement is an advantage in one sense, as, because of it and of the broad expanse of water in which the artificial channel is made, the velocity of the currents is less and the consequent amount and movement of matter in suspension is small. These circumstances favor the permanence of this channel, a thing to be rarely expected of a dredged channel, unless regulating works be added.

The idea of such training walls in this harbor has been considered, but their presence would be strenuously opposed by the great number of navigators in smaller vessels, especially sailing craft and light draught steamers, which need not go in the deep channel and have occasion to be outside of it. The constant use of the deep channel by the large screw steamships of oceanic lines helps greatly to keep the channel clear, and while it may be necessary to occasionally redredge it in one or two small spots, the annual repair of this kind is small, and less than the interest on the first cost of long training walls which would also need occasional repairs.

It has been objected that the dredges are much in the way while at work. This is true, but the objection lessens in force as the width of

the channel increases and approaches its limit. On the other hand the dredges are entirely out of the way when not at work, whereas training walls in this locality would be a constant obstruction and danger to navigation, and serve as places of lodgment for the ice which sometimes though rarely fills the harbor.

Of late years the dredging has been done entirely by contract. The old, original dredges were long ago sold. They needed much and expensive repairing to keep them up to their full capacity for work, which was small, not exceeding 400 to 600 cubic yards of material a day. They were, moreover, a constant subject of worry and contention for the engineers who wished to have the crews of the best material without regard to their politics. The politicians took a different view and sometimes carried their point against the engineer. The large dredges of the present day can do five times as much work in good weather, and can work in much weather which would oblige the old style dredges to go to a harbor for safety. The price of the dredging has been also very greatly reduced, viz., to about one-third of what it formerly was.

The disposition of the dredged material is a matter of great moment, especially when the quantity runs up into the millions of cubic yards. The important point is to put it where it will do no harm in the future. If it can, moreover, be put where it will actually do good, there is a great gain. The general rule adopted here has been to prescribe that material taken from the channel may be dumped in the bay, but not nearer the dredged channel than a mile east of the Craighill division and a mile south of the Brewerton division. By this disposition of the material and its spread over a large area, no obstruction to navigation is caused and there is small probability of any portion of the deposited matter working back into the channel or becoming otherwise hurtful. Moreover, from the space this mass occupies an equal volume of water is kept out and is thus compelled to take its place in the excavated channel.

In excavation no effort has been made to give with the dredge any specified slopes to the sides of the dredged channel. To require anything of that kind would cause contractors to raise their prices, even if the whole work were contracted for at once. In this case, where the work goes on by successive additions to the width from year to year, under the present system (or rather want of system) of intermittent and insufficient appropriations, the best way has been in any one season to make the sides of the excavation practically vertical. As the side slope slips in, the dredge keeps the bottom clear until the natural slope in the water is gradually taken. If a particular slope were assumed in the beginning and required of the contractor, natural causes would be almost sure to change it and redredging be required if the prescribed slope were too great; or, if the prescribed slope were less than the natural, in order to insure stability, more material would be generally taken out than necessary, and at a greater cost than by the method pursued.

It will be noted that the channel is in straight lines. The consequence is greater ease of navigation. The channel here is well marked by range lights. The navigator coming up the bay soon finds the range of the Craighill division. He follows it in a line very nearly north and south until he is on the range of the Cut-off, along which he continues in another straight line until he is on the range of the Brewerton division, and he is thus brought within sight of the city and near well-known land-marks.

Objection has been made that some dredging will be needed occasionally to keep this channel in good condition. The length is about eighteen miles. The annual cost of dredging will not exceed from $30 000 to $50 000. This does not seem a large sum when the advantages of the improvement are taken into account and consideration is given to the need of keeping in repair all artificial highways, such as railroads, canals, common roads, streets, etc. The main need for annual repairs at this time is caused by the narrowness of the channel. When a great steamer shears a little and strikes a glancing blow against one of the banks of this canal dug in the bottom of the river and without revetment, her sharp prow, like a plow, makes a huge furrow and throws a large mass of mud into the water way.

It may not be uninteresting to note that about three-quarters of a century ago Admiral Cockburn found an obstruction to the navigation of his fleet towards Baltimore at an insignificant little place called Fort McHenry. We do not propose to allow any hostile admiral ever again to get so near to Baltimore. The outer line of defense is now that of North Point to Bodkin Point, where earthen batteries will be located, with a turret or two on an artificial island at "Seven-Foot Knoll," which can be readily made from the material to be still taken from the channel. The inner line will be that of which Fort Carroll is the center.

Before leaving the Patapsco it occurs to me to mention and call attention to one other spot about midway between Forts Carroll and McHenry where our glorious air, "The Star Spangled Banner" of Frank Key was born. Long may that banner wave, not only on the Patapsco, but over this whole broad land,

"The land of the free and the home of the brave."

James River, Virginia.

The portion of this river which has been the subject of improvement is that below the City of Richmond, the head of tide-water and of navigation. The improvement was regularly undertaken by the United States in 1870. A small sum had been previously expended, but with little advantage. In addition to the appropriations from the general treasury the City of Richmond has contributed her money liberally towards the expense of this work, in which she is vitally interested.

When the improvement was begun by the United States, navigation was much hindered by sunken vessels, the remains of military bridges, and other obstructions put in the river during the late war to prevent the national fleets from approaching too close to Richmond. There were also natural obstacles. Rockett's Reef, at the border of the city (Plate XXXV), and Richmond Bar, a short distance below, had each only 7 feet of water at mean low tide. From Warwick Bar (where the depth was 13 feet) to Richmond the channel was crooked and obstructed by dangerous rocks and ledges. The Dutch Gap cut-off, which now saves 5½ miles of difficult navigation, was not then open (Plate XXXV).*

The original project was to secure a depth of 18 feet at full tide, corresponding to 14.5 at low tide, with a channel width of 180 feet from Harrison's Bar to Richmond Docks, 37½ miles above, the excavation in rock to be 18½ feet at full tide. This project was well advanced when Congress, in 1884, ordered that subsequent operations should be directed to the attainment of a depth of 22 feet at mean low tide from Richmond to the sea (Plate XXXVI). This extended project is an expensive one, as it requires the removal of a large quantity of rock. In carrying it out, the width proposed for the channel is 400 feet from the sea to City Point, where the James receives the waters of the Appomattox; 300 feet from City Point to Drewry's Bluff,† which is about 7 miles below the city, and 200 feet thence to Richmond. The change from the former to the latter project was a great one and rendered useless much of the work already done. Very little money has thus far been available for the new scheme.

The methods employed for improving the river have been dredging, rock excavation, and the contraction and regulation of the water-way by training walls and wing dams. The rule has been to use the small and inconstant supply of funds in attacking always the shoalest places, with a view of obtaining the greatest benefit to commerce in the shortest time.

The present condition of the river may be known from the fact that vessels drawing over 16 feet can now be loaded to that depth at the city and go out. (Plate XXXVII.) The United States ship Galena is expected up soon and will not on this occasion stop at Drewry's Bluff.

As is usually the case, the main trouble has been with the straight and wide reaches of the river, and at points where there is a cross-over of the channel from one side to the other, due to change of curvature of the river or to some local cause. By far the most expensive part of the improvement has been and will be, the few miles from Richmond

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* See also Coast Survey Chart 402, showing the Appomattox and the James Rivers above City Point.

† The line A B on Plate XXXV, is the same as the line A B on Plate XXXVIII. If one line were placed over the other, the two plates would show the relative directions of the James and Appomattox Rivers.

to Drewry's Bluff. This is to be attributed to a great extent to the physical characteristics of that portion of the river, and chiefly to the necessity of removing a great quantity of rock.

The rock in this river is granitic, partly in beds and ledges, and partly in detached boulders. There is much of it which has never been compacted or has become disintegrated, and is so soft as to be readily removed by a strong dredge. Before the war the City of Richmond had done some dredging of sand and mud from the worst shoals and had taken out some loose rock. The dredged material was not disposed of to the best advantage, being often dumped where much of it came back into the channel again.

The first operation undertaken by the General Government was that most needed—the removal of the artificial obstructions. This was quite a tedious though a comparatively simple job. The worst place was at Drewry's Bluff, where the natural channel had been completely closed by sunken vessels. A narrow channel was formed in an unnatural position close to the left bank. This arrangement brought vessels better under fire from the powerful batteries on the bluffs of the right bank. The remains of a number of steamers (some iron-clad) and sailing vessels, as well as cribs filled with stones, were removed at Drewry's Bluff by the aid of the diver, explosives, dredging and hoisting apparatus. Vessels and the remains of military bridges on piles and cribs were taken out at other points.

At the same time the opening of the Dutch Gap cut-off was systematically begun. It had been attempted during the war under the direction of General Butler, but the work was too much hindered by hostile guns at that time.

By making this Cut-off, the improvement of Trent's Reach, one of the worst places in the river, became unnecessary, and a distance of $5\frac{1}{2}$ miles was saved to every navigator.

The next steps in the improvement were the beginning of the construction of a system of training walls and wing dams to give an increased depth of water in a narrower and regulated channel, with the use of the dredge to remove the worst parts of the worst shoals, in order to give immediate relief to navigation, which could not wait for the slower operation of regulating works. This great use of the dredge gives certainly quick results, but it is an expensive method of procedure, as a freshet of a few days' length has often obliterated the work of months of one or more dredges. But the people will have it so.

Whenever any part of the meagre appropriations could be spared for the construction of regulating works, some progress was made with them. Besides this function, these works gave places behind which the dredged material could be deposited, a matter of very great importance. It has saved very much expense when such material could be disposed of by being carried by water from the dredges, and dumped immediately

from the scows behind the training walls and spurs. When it becomes necessary to remove it to the banks of the river and rehandle it, the expense is much increased.

The adoption of a system of regulating works requires the engineer to decide at any particular locality, whether to rely on longitudinal training walls or wing dams, or a combination of both. In some respects, the longitudinal training walls would be preferable, if it could be always known with certainty where to put them, and have the best results for the money spent. In most cases it is impossible to decide this point with absolute precision. Even at the mouth of the Mississippi River, when the problem was in the hands of so able a man as Mr. Eads, he found it expedient to use spurs in addition to his original training walls.

It may be said of longitudinal training walls, that if they are put in deep water they become very expensive. If placed too far from the channel to be improved, their effect is insufficient. If placed too near, they are likely to be undermined.

On the James River, the arrangement of the regulating works has been arrived at in the following manner. After the surveys and maps were made and due investigation was had of the requirements of the case, the formulas told what should be the width of channel of an assumed depth and shape, which a river with the characteristics of the James would bear. Inasmuch, however, as a river is seldom the same creature for two successive days, and the variations of regimen in a year are large, there are, for these and other reasons, elements which cannot be accurately taken into account by the rigid rules of mathematics. In other words, the best hydraulic formulas give but approximate results. Woe to the man who follows them strictly!

Under these circumstances, the method of improvement must be to some extent tentative. If one bank of the river is firm and the shape of the reach favorable, spurs may be run out from the other bank, gradually, to nearly the length which would be needed to give the contraction indicated by the formulas. Observation for a year or more may show the need of lengthening all the spurs to produce the desired effect. Or, it may be found they are all too long and must be cut off. Or, it may be necessary to lengthen some and shorten some.

If the bank towards which the spurs run is liable to erosion, it must be reveted, or a training wall be built in front of it. Sometimes it may be best to run spurs from both banks simultaneously. As far as injury from undermining is concerned, the danger for spurs is less than for training walls, as only the ends of the spurs are thus exposed. When the proper length for the spurs has been determined, their ends may be connected by a longitudinal wall.

Longitudinal walls, especially if high, interfere more with the tidal reservoir, the retention of which in full is of much importance. The

spaces between the spurs are readily filled by the rising tide, and emptied on the ebb, but the swirls at the ends of spurs are often disadvantageous.

The due regulation of the height of the works is a most important matter. If too low, they control the currents too little. If too high, they obstruct them too much, especially in time of freshets or when there is danger of ice-gorges.

When a river is running one way all the time and its vertical variation is not great, the location of one spur or jetty may not be difficult. When the river is tidal and, moreover, subject to considerable freshets, the problem becomes more complicated. When more than one, a whole set, must be put in, the thing is still more difficult.

As the outer part of a spur or jetty is what mainly does the work of scour, that part should be quite or nearly normal to the thread of the current near its end. The direction of the part shoreward should be so arranged as not to check the water too much or produce scour where it would be unnecessary or hurtful. The spur should sometimes be not straight, but curved or composed of two or more straight branches.

The distance apart of spurs should depend upon the work to be done and the means of doing it. In a sluggish stream, for instance, or a sluggish part of a stream, other things being equal, the spurs of a set should probably be nearer together than in a rapid current. And in the same case more of them would be needed at the lower end of a set. The distance apart should also vary with the nature of the material to be scoured (the velocity being constant), the distance apart being lessened as the weight of the material to be scoured is greater.

The height should be regulated to make the spur do its work at the stage when work is necessary, and not be too much of an obstruction or produce too much overfall at other stages.

On the James River, as elsewhere, the question has more than once arisen as to the relative use, in time and extent, to the best advantage to the improvement of the dredge and regulating works. When public attention is sufficiently drawn to a river or harbor to cause an appropriation of money for it from the United States Treasury, quick results are demanded. The engineer is then forced to use the dredge, although he may be sure that the work in whole or in part must soon be done over again, which need not have been the case if there had been money available to construct a proper system of regulating works and the interests of navigation had permitted the delay. If he can proceed with his regulating works while the dredging is also in progress, he is a fortunate man, as he may then expect the former to hold the channel as the dredges make it.

The matter of the comparison of the costs:

First.—Of the making of a channel by dredging and its retention by the occasional use of the same means without regulating works ;

Second.—Of the cost of a complete system of regulating works with the interest thereon yearly, without resort to dredging ; and

Third.—Of the combined use of the two ;—is a thing to be carefully considered in every case that arises.

It sometimes happens that owing to the rapidity of sedimentary deposition or the exposure of the locality the dredge cannot be used to make and hold the required channel. Auxiliary works are then a necessity if the channel is.

It is also rare, in considering the second alternative of dispensing with the dredge, that this is possible; because, while in dredging one may know with certainty the place where the material removed is to be deposited, there is always some uncertainty as to the effect of regulating works, and the dredge may at last be needed to attain it fully and perhaps to remove a new shoal caused by the works themselves. The case may also arise where, while regulating works will hold a channel when once made, the material is such as to require the use of the dredge to make it altogether or to hasten its completion.

He is a wise and fortunate man who knows how to use both expedients to the best advantage in aid of each other and has ample funds at his command to do what is right at the proper time, instead of being obliged to do the best he can with limited means and time at any particular stage of the improvement.

Very important lessons can be learned in this connection from the experience with the successful work of Mr. Eads at the mouth of the Mississippi River. The following statements are interesting, and come from a reliable source:

"The distance between the jetties proper is 1 000 feet. Wing dams or spurs were built in order to contract the water-way, and thereby hasten the channel deepening. The maintenance of the width of water-way between the main jetties (1 000 feet apart) was not contemplated, but they were placed that distance apart, so that by the auxiliary works they could be secured against undermining.

"From the first it was Mr. Eads' idea to close Grand Bayou ; therefore, the ultimate width of water-way through the jetties was to correspond with the average width of the pass above that bayou, which is 700 feet, in order to secure the same average depth as existed there, viz., 30 feet. Again, there were places in the channel where the material was tough clay and did not scour readily, and at such places the water-way was reduced more than at any other.

The jetties were commenced in June, 1875—the first spurs were built in June, 1876 ; and from time to time since spurs have been built. The inner jetty, on the east side, was commenced June 20th, 1882, and on the west side in February, 1886. There was no special order in which these works were built ; that is, while the inner jetty was being built on the east side, spurs were built on the west side, etc. The building of inner longitudinal works was not contemplated at the time most of the spurs were built, but later on, when it was found that considerable sand was thrown over the main jetties during storms, and worked its way into the channel, the idea of the longitudinal works (inner jetties) to prevent

this, also to prevent the escape of water at high tide, and at the same time to answer the purpose of a wing dam at every point, was adopted.

"March 24th, 1878, a section of the west jetty, 110 feet in length, commencing 430 feet from the upper end, was undermined. Piles in the row nearest the channel and at the sea end of the inner east jetty, have, from time to time disappeared, but aside from that mentioned, there has never been an undermining of an entire section. In 1876 and 1877, much dredging was done by three ordinary dipper dredges (the large dredge was not built then,) and at the same time spurs were being built. These two means were used conjointly, and places which did not readily yield to the scouring action of the current were dredged. I am of the opinion that, in time, the channel would have developed without the aid of dredging ; it was only to hasten results that it was resorted to—but without the spurs, the required channel could not have been maintained even with the aid of dredging, had the water-way remained 1 000 feet in width ; therefore, I consider that spurs were absolutely necessary, but dredging was not.

"As finally completed, that is, after expensive alterations had been made, the cost to Mr. Eads of the large dredge was $150 000. On a cash basis it is probable that the boat could have been duplicated for $100 000 at least.

"The boat was in commission from November 22d, 1877, to February 22d, 1883, during which time she was laid up one year on account of repairs, and because of the epidemic of 1878; therefore, she was actually in commission five years and three months, and in this time she worked three hundred days of ten hours each. About three-fifths of this time she worked in the channel between the jetties, and the rest of the time at different localities, viz., head of South Pass, above Goat Island, near Grand Bayou, and on lumps beyond the ends of the jetties.

"No dredging has been done since February 22d, 1883. At times during the past five years it has seemed probable that the services of the large dredge would be required, but that expedient has been avoided by building spurs—the prospects of her being used are no greater now. While some have often favored dredging, Mr. Eads strongly opposed it; and if, with the works as they are at present, the channel should shoal to less than the required dimensions the dredge would be used. The conditions of the channel being influenced as they are by the stage of the river, and that from year to year being a variable element, it is hard to predict what may take place from what has taken place.

"None of the spurs have ever been removed—the channel ends of some have been undermined and washed out.

"The width between the longitudinal or inner jetties is 675 feet, but at places the spurs reduce the width of water-way to 570, 590, 600 and 630 feet."

It would be interesting to consider the effects of the Dutch Gap Cut-off on the regimen of the James River. It is hoped that this and some other matters may be brought out in the discussion of this paper by two of our members who have been associated with me in dealing with this stream. The advantages of this Cut-off have led some to advocate earnestly others at Jones' Neck and Curl's Neck (Plate XXXVIII). It has also been seriously proposed to make a canal from the James, near the Dutch Gap cut-off, to the Appomattox at Port Walthall, and thus

avoid the circuitous navigation from the Dutch Gap to City Point by the present route.

While on the subject of cut-offs on the James, reference may be made to a successful improvement of the Pocomoke River, Maryland, shown on Plate XXXIX, in which a cut-off from *a* to *b* (and really to *c*) was a prominent feature, the channel having previously been by the circuitous way of *f* and *h*.

The dredge alone was used, without auxiliary works. The dredged channel has stood well. The cut-off was through a thick cypress swamp. Explosives were to some extent advantageously used on the larger stumps. My successor in charge of that improvement is making another cut-off from *d* to *g*.*

Appomattox River.

The portion of this river below Petersburg to its confluence with the James is short, being about twelve miles in length, but it has been the subject of much torment from engineers for about two-thirds of a century. This was done mainly by the expenditure of the money of the City of Petersburg. The United States engaged in the work very little until after 1870.

The map (Plate XXXVIII),† shows the condition of the river in 1855. Below the Point of Rocks nothing has ever been done, as the depth of water upon that part of the river has sufficed for the navigation. From Petersburg to Point of Rocks the river flowed through or was bordered by low, marshy ground on one side or the other. It is stated that before the war vessels drawing 9 feet could reach the wharves of the city. This was at high water, and indicated a depth of about 6½ or 7 feet at low water. During the war it was considered expedient by the Confederate authorities to close the river entirely to navigation by material obstructions. After the war the authorities of the city removed these obstructions in great measure, but the river was found to be in very bad condition, as on more than one of the shoals there was a depth of but 3 feet at low water. The navigation was down the main river to the upper end of the little gut at *a*, through which vessels passed to *b* and so out by *c* and *d*.

Besides the shoals mentioned, there were other things to be taken into account in a project of improvement. The harbor of the city was the narrow portion of the river in front of it, and this was constantly filling with sand and mud brought down from the part of the stream above Petersburg. On the eastern border of the city was a small stream

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*For a description of the James and much other interesting matter see specially the annual report of Chief of Engineers, 1882, Part I, pages 860–898; and annual report for 1875, Part II, pages 74–80.

† See Coast Survey Chart 402, showing Appomattox and James Rivers together.

called Lieutenant Run, emptying into the river at right angles at *m* and bringing down in freshets a large quantity of detritus. Below its mouth the channel after crossing over followed the left bank, but in the concave, at *e*, there was some dangerous rock. The other difficulties below this point were the shoals of sand and mud.

In 1869 the engineer of the city prepared a project which contained the following features, as stated by himself:

"My plan is to straighten the river and ease the curves or bends where it can be done to advantage and with not too great an expense, and to confine it for the whole distance, from Petersburg to the Point of Rocks, by artificial banks, wherever they are needed, to one channel, expanding regularly in width from 200 to 500 feet, so as to give ample room for the freshets and flow of the tides.

"To make this plan as perfect as I conceive it could be made, I did not hesitate to abandon the present channel of the river almost entirely, for with the construction of about a mile of new channel I shall shorten the river nearly half a mile and conduct it into the old south channel, which, no doubt, was anciently the true channel of the river. In the old works it had been shut off repeatedly, but never perfectly, and when I took charge of the improvement fully half the flow of the downward water passed into it. One has only to see this beautiful body of water, with its comparative straightness, and bold southern shore, to be satisfied that with little expense it can be made the finest part of the river. The north shore or bank of this channel and everywhere else, where the bank is needed to confine the river, will be made of the material dredged from the river to obtain the necessary depth of water.

"I am satisfied from the experience of this river, and from the results shown in other rivers, that the increased impetus which this shortening and narrowing of the river and confining it to one channel must give to the freshets and tidal flow, will keep the bed of the river so scoured out that very little dredging will be required, except to strengthen the banks or make the navigation deeper."

When the United States took the work in hand in 1870, it was found that the means then used to confine the river was a high fence, made of sheeting sustained by round piling and wale pieces. About two miles of this fence had been finished. The engineer stated it had been located up to that time "wherever there is water or very low places through which the current might be diverted." In the first freshet after the building of the fence the river carried away a large part of it near $f^1 f$, as well as injuring it elsewhere. In addition, a coffer-dam had been built at much expense to make a straight channel from *g* to *h* through the rock. This coffer dam had been seriously damaged. Some dredging had also been done in the line of the channel in the Puddledock Marsh.

It had been the expectation of the engineer of the city that if the fence were built across the river at $f^1 f$, and a small cut made with the dredge through the Puddledock Marsh, the river itself would in the first freshet widen and deepen the Puddledock cut as much as was needed. This expectation was based on what had taken place on a smaller scale

at $f\,l$, where many years before "a straight cut through an island was made with shovels, the length of the cut being about ¾ of a mile, its width 10 feet at the water-line and the depth 4 feet. After this was done the channel on each side of the island was dammed up and the cut was left to the action of the river. The cut was scoured out to a navigable depth, as was stated, in about thirty days."

The river would not act in this way in the proposed Puddledock channel, but seemed to prefer the old route to the sea. The reason was probably that the island through which the cut was made in the first experiment was nothing but alluvium, whereas the soil in the Puddledock Marsh was filled with roots and stumps of cypress and other trees, and much of the material was so hard as to require subsequently a strong dredge for its removal, and explosives had to be used on some of the stumps before the dredge could start them.

The funds of the city were by this time nearly exhausted, the appropriation of the United States was small, and the pressure for immediate results was, as usual, strong. The question was what to do. The fence was abandoned, as it was too weak and not well located for its object, and the channel it was expected to provide was much larger than the river would maintain.

An examination of the history of the river showed that the "South Channel" from s, the lower end of the proposed Puddledock cut, had been quite constant and its right bank was permanent and bluff. It was decided to improve this stretch.

It was in contemplation at one time to connect this South Channel with the channel then in use by making a cut on or near the line $s^1\,s$ or $t^1\,t$, the left bank of the river above that proposed cut being also bluff and apparently permanent for some distance up-stream. As an alternative the plan of making a cut from a, past c to k, was discussed.

But as so much progress had been made on the Puddledock cut, and for other reasons, it was decided finally to open that cut with the dredge and close the river at $f^1\,f$, which was done. The idea was considered of putting into the closing work at $f^1\,f$ tide gates, automatic or otherwise, to admit the flood water by that means, and to cause the same water on the ebb to go through the Puddledock cut and the South Channel, but this was not carried out.

The advantage of making the channel take direction from g to h near the city, has been constantly recognized, but its expense, as it is in rock, has never allowed this to be done. The course of Lieutenant Run was changed so as to have it come into the river lower down and at a very acute angle. Its detritus was thus deposited in the space $g\,h$ where the straight cut was expected to be, but out of the used channel.

Some spurs on the left bank opposite the city, and frequent use of the dredge belonging to the city, are depended on to keep "the harbor" clear. Some of the rock has been taken out near e. The river below

the town to the upper end of the Puddledock cut has been regulated by the training walls and wing dams to give a channel 100 feet wide and 12 feet deep at high water. Spurs have also been put on the left bank of the South Channel below the Puddledock cut to maintain the deepened dredged channel.

The improvement has been a successful application of the use of training walls and spurs. In the beginning, before the regulating works were put in, it was necessary to use the dredge constantly. As the regulating system progressed, the use of the dredge was continually less, until now it seldom appears upon the river. A quick freshet sometimes causes a little shoaling which the regulating works would remove in a moderate time, but the dredge is then brought into requisition to clear away the obstruction at once.

In making the cut through the Puddledock Marsh, artificial banks have been made of the dredged material. To do this, it could not, of course, be emptied in the usual way from dumping scows. A cheap and simple arrangement was used. On a small decked scow alongside an ordinary dredge a hopper was put up as high as the dipper of the dredge could lift the material. While the material was thus dropped into the hopper from the dipper of the dredge, a stream of water was at the same time pumped into it. The mixed mud, sand, gravel and water ran off together through a cheap wooden trough to form the artificial bank. This arrangement is only mentioned because it preceded, as far as the writer knows, the use in this country of the methods which have been extensively used on the Potomac Flats, at Oakland, California, and elsewhere for conveying dredged material long distances. On the Appomattox it was only necessary to have a trough some fifty feet long, which was readily carried from the side of the hopper. The same idea was also used a little later on the Wicomico River, near the town of Salisbury, Md., where the conveying trough, about 175 feet long, was supported on floats.

The artificial banks on the Appomattox and elsewhere are planted with willows, and these soon become a very dense growth, and successive crops have been extensively used for the manufacture of fascines, mattresses and hurdles which have formed parts of the regulating works.

The Appomattox is by nature a narrow stream, and the regulating works in some places make it still narrower. Some navigators find fault with the width, and it has been proposed to build a dam above Petersburg, and turn the whole river into the low grounds to the west of the present left bank, with another connection with the old river some two or three miles below. This scheme involves also the removal of all the existing works of contraction, and depends on the tidal inflow entirely for the channel below the city. It is expected in this way to be rid of deposit in "the harbor" (which would depend for its water on the tidal inflow entirely), and to have a wider channel below independent

of freshets. The promoters of this project are allured by some of its attractive features, but lose sight of serious objections which may be brought against it.

Wicomico River.

This river on the eastern shore of Maryland empties into Chesapeake Bay, nearly opposite the mouth of the Potomac. It is not a large stream, and flows through a flat country; its banks are to a great extent marshy and its bed muddy. Its width is great in comparison with other elements of the regimen.

When the river was examined some years ago with a view to its improvement by the United States, it was found that navigation to Salisbury, one of the largest towns on the eastern shore, was prevented by a bar a short distance below the town, composed of mud, sand and sawdust, on which there was a depth of less than a foot at low water. A gauging of the stream showed that it was capable, with proper treatment, of having a channel 7 feet deep at low water and from 75 to 100 feet wide. Contraction of the width was the remedy with the help of the dredge.

Steps were also taken to prevent the further emptying into the river of the large quantities of sawdust thrown off by the extensive mills of the town. This material, sawdust, soon becomes water-soaked, and falling to the bottom and mixing with the sand and mud makes a tough bar not easily movable by water at low velocities.

The map (Plate XL) shows a part of the area over which the improvement has been carried. A new river has been made. The contracting dikes were cheaply built of the timber of the country, being brought a little above the line of high water. The loss of water through the side channel behind Caucasin's Island was prevented by the dike. The dredged material was placed behind the dikes or in side channels and hollows whence the water was driven into the channel. In some cases the material was troughed 175 feet in the way mentioned in the part of this paper referring to the Appomattox River. The artificial banks have in some parts been planted with willows, etc., and have become well consolidated. The dredged channel has been carried some 3 000 feet below the limit of the map and the material utilized as explained, but, as usual in such cases, there has not been money enough provided for completing the lines of dikes.

It is to be expected that some dredging must be occasionally done in an artificial channel like this (specially while the diking is incomplete), which should be classified as repairs. To such treatment all the works of man are liable. An improvement like this is also most important in increasing the healthfulness of a large community by facilitating the drainage of a wide area of country.

This improvement has been mentioned because it has been one of the localities ridiculed in Congress and elsewhere. The general outline of

the matter is the following: This river has the town of Salisbury at its head. Some years ago no steamboat, and in fact no boat of any size, could reach the town, inasmuch as there was a shoal below it on which, at low tide, there were only about 12 inches of water. Now there are 7 feet, and at high tide 3 feet more. What is the effect of this improvement, which has cost about $30 000? It is to cheapen freights, and bring to Baltimore by a free and natural highway much trade which formerly went elsewhere by an artificial one. Salisbury and the country immediately tributary to it have also been greatly benefited. What benefits Baltimore, a great national seaport, is a thing of national interest.

The method of the improvement of the Wicomico is precisely the same as that employed on the Clyde, in Scotland, on which millions have been expended to give sufficient depth of water for the largest ships, and to this end the dredges have dug through shoals where formerly the depth of water was measured by the inch. In all such cases, where there is an unlimited reservoir of water to be drawn upon, like Chesapeake Bay and the ocean, the depth which can be had is limited only by the amount of money available.

Many persons suppose that the Clyde is a grand, broad river. This is a great mistake. The Wicomico compares favorably with the Clyde as to width, and can be made as deep for a tithe of the money. Some people will say Glasgow, on the Clyde, is a great city, and Salisbury, on the Wicomico, is insignificant. When Glasgow was as young as Salisbury she was quite as insignificant. The greatness of Glasgow has grown with the improvement of the Clyde, and so might Salisbury grow with the Wicomico.

Ridicule has also been cast upon the improvement of Corsica Creek, on the eastern shore of Maryland, by some who never saw it, and do not know that this creek, which might be properly called a river, is actually as large as many rivers in Great Britain and on the Continent of Europe, where millions have been expended for their improvement, and with the most valuable results.

Nobody doubts, I suppose, the propriety of the improvement by the United States of a great river like the Mississippi or the Columbia, but these streams are not great throughout their whole lengths. They become smaller towards their heads. Where will the line be drawn where the general Government shall cease its work of improvement, and who will then continue it? If difficulty be found in drawing such an artificial line, will you reject as a proper subject for improvement by the United States a river that is comparatively small but larger than the upper part of the Mississippi or the Columbia, and which with its connections, the ocean, has a common interstate and European outlet?

Sometimes also we hear sneers at the idea of improving streams of considerable size for the purpose of facilitating and cheapening the

bringing to market of millions of feet of lumber of the most valuable and various kinds. The sneering ones say that logging is not navigation, forgetting or not knowing that the timber which is brought down many streams not navigable by boats is one of the most valuable products for the commercial uses of the whole country. In its movement to a market it has the peculiar property of being its own carrier and does not need the help of any other by land or water.

In truth, an annual River and Harbor Bill of fifteen or twenty millions of dollars is not unreasonably large for such a country as ours, considering its immense extent, its rapid development and other circumstances. Though such a sum looked at by itself without regard to its object seems a large one, it is actually small in view of real needs, and appears almost insignificant in comparison with those of other nations of less extent and population, much of whose wealth and power come from the effects of just such works of internal and coast improvement as such expenditures have allowed.

A number of streams on the eastern shore of Maryland have been improved by the United States after methods similar to that referred to for the Wicomico. One of these is the Elk River at Elkton on the line of the Philadelphia, Wilmington and Baltimore Railroad. This is mentioned here in order to call attention to a detail of which it is the first instance in this section. In 1883 the old regulating dike was found to be so decayed as to require renewal or removal. This dike had originally been built as a bulkhead behind which dredged material might be conveniently placed, and thus prevented from going back into the channel, as well as to contract the water-way. It had always been expected this longitudinal dike of timber would decay at or near the water line, and must then be renewed or removed, or the earth behind it, having had time for consolidation, be sloped back and formed into a new bank for the stream. It had been found also that vessels use the piles of a timber revetment to fasten their lines upon, and frequently pull them over and make a hurtful obstruction. It was concluded, therefore, to remove the old dike and slope back the bank, which was done, and the river was thus left with a new but permanent, though artificial bank.

CONCLUSION.

It had been the wish of the writer to add a brief account of the improvement of the Cape Fear River, but want of time and other causes have prevented. The problem there has been a very interesting one, the main points having been the closure of one mouth of the river, which was about a mile wide, and the use of the dredge and regulating works at other points. The improvement covered a stretch of about 25 miles from Wilmington to the ocean.

The results of the work on the several rivers referred to in the preceding paper may be briefly given as follows:

Patapsco.—Improved position, length and width of the channel; increase of depth from 18 feet at mean low water to 27 feet at the same stage.

James.—Improved position, length and width of channel; increase of depth from 7 feet at mean low water to 13½ at the same stage.

Appomattox.—Improved position, length and width of channel; increase of depth from 3 feet at mean low water to 9 feet at same stage.

Wicomico.—Improved width of channel and increase of depth from 1 foot at mean low water to 7 feet at the same stage.

Pocomoke.—The object was not an increase of depth but of width and direction, both of which have been attained.

Cape Fear.—Improvement in depth on sea-bar from 9 feet at mean low water to 14½ at same stage; increased depth to Wilmington from 9 feet at mean low water to 16 feet at same stage.

The Plates accompanying this paper are :

Plate XXXIV.	Patapsco River and Baltimore Harbor.
" XXXV.	The James River from Richmond to Jones' Neck.
" XXXVI.	Profile of the Channel of the James River from Richmond to the Sea.
" XXXVII.	The James River from Richmond to Falling Creek.
" XXXVIII.	The Appomattox River.
" XXXIX.	The Pocomoke River.
" XL.	The Wicomico River.

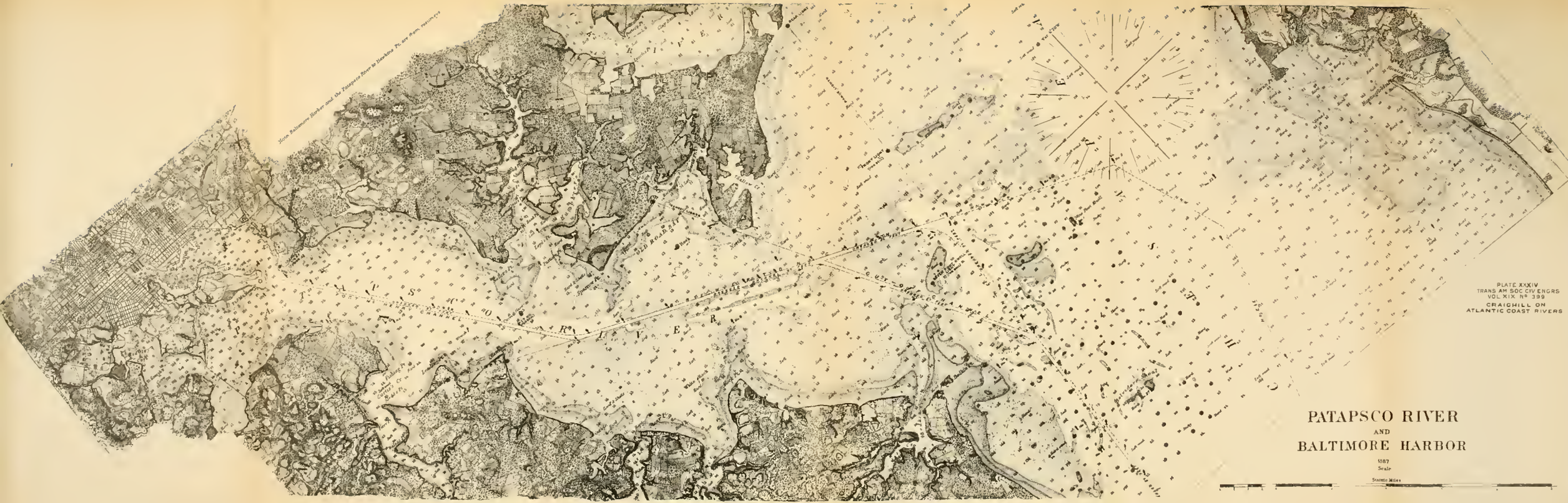
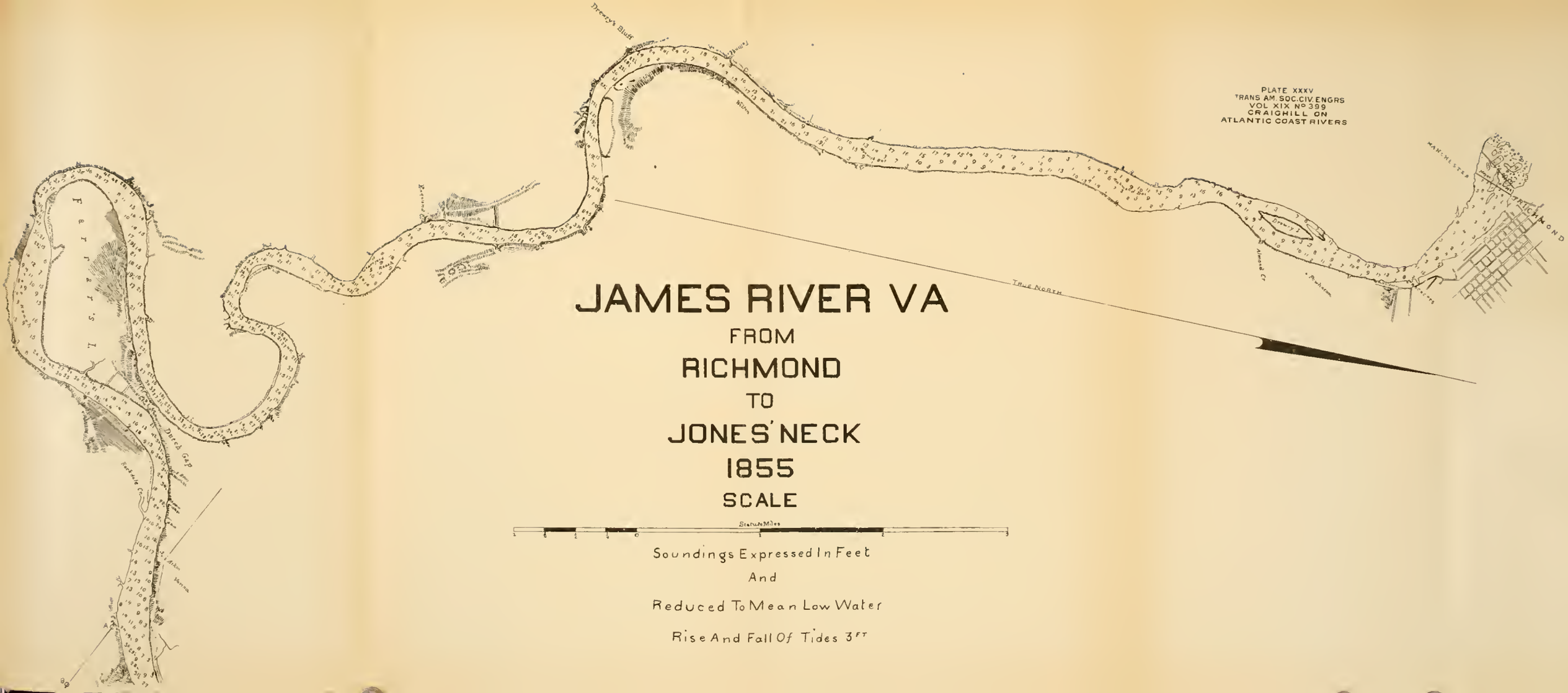

PATAPSCO RIVER
AND
BALTIMORE HARBOR
1887
Scale
Statute Miles
PLATE XXXIV
TRANS AM SOC CIV ENGRS
VOL XIX Nº 399
CRAIGHILL ON
ATLANTIC COAST RIVERS

PLATE XXXV
TRANS AM. SOC. CIV ENGRS
VOL XIX Nº 399
CRAIGHILL ON
ATLANTIC COAST RIVERS



JAMES RIVER VA

FROM

RICHMOND

TO

JONES' NECK

1855

SCALE

Statute Miles

Soundings Expressed In Feet

And

Reduced To Mean Low Water

Rise And Fall Of Tides 3 FT

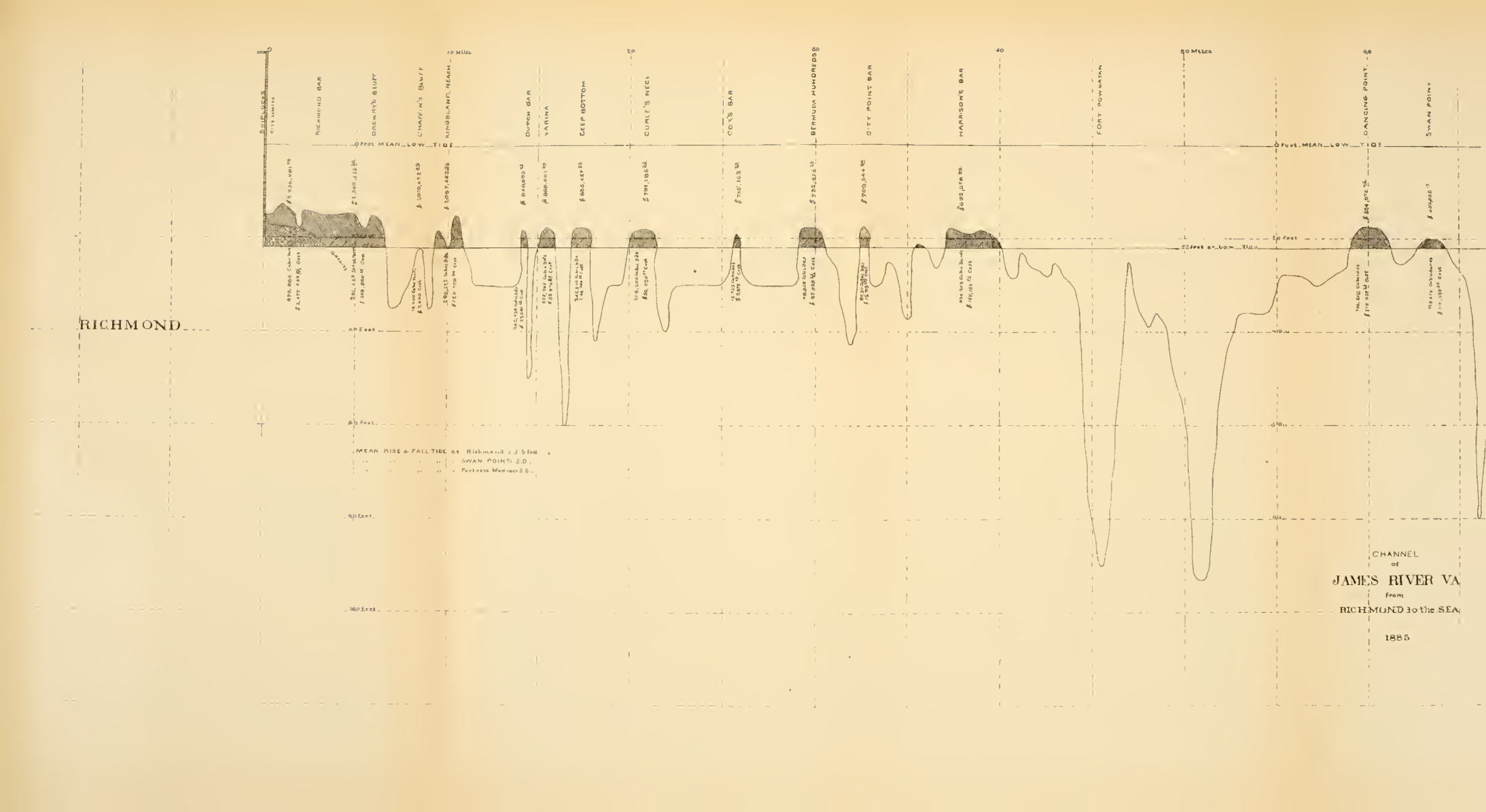
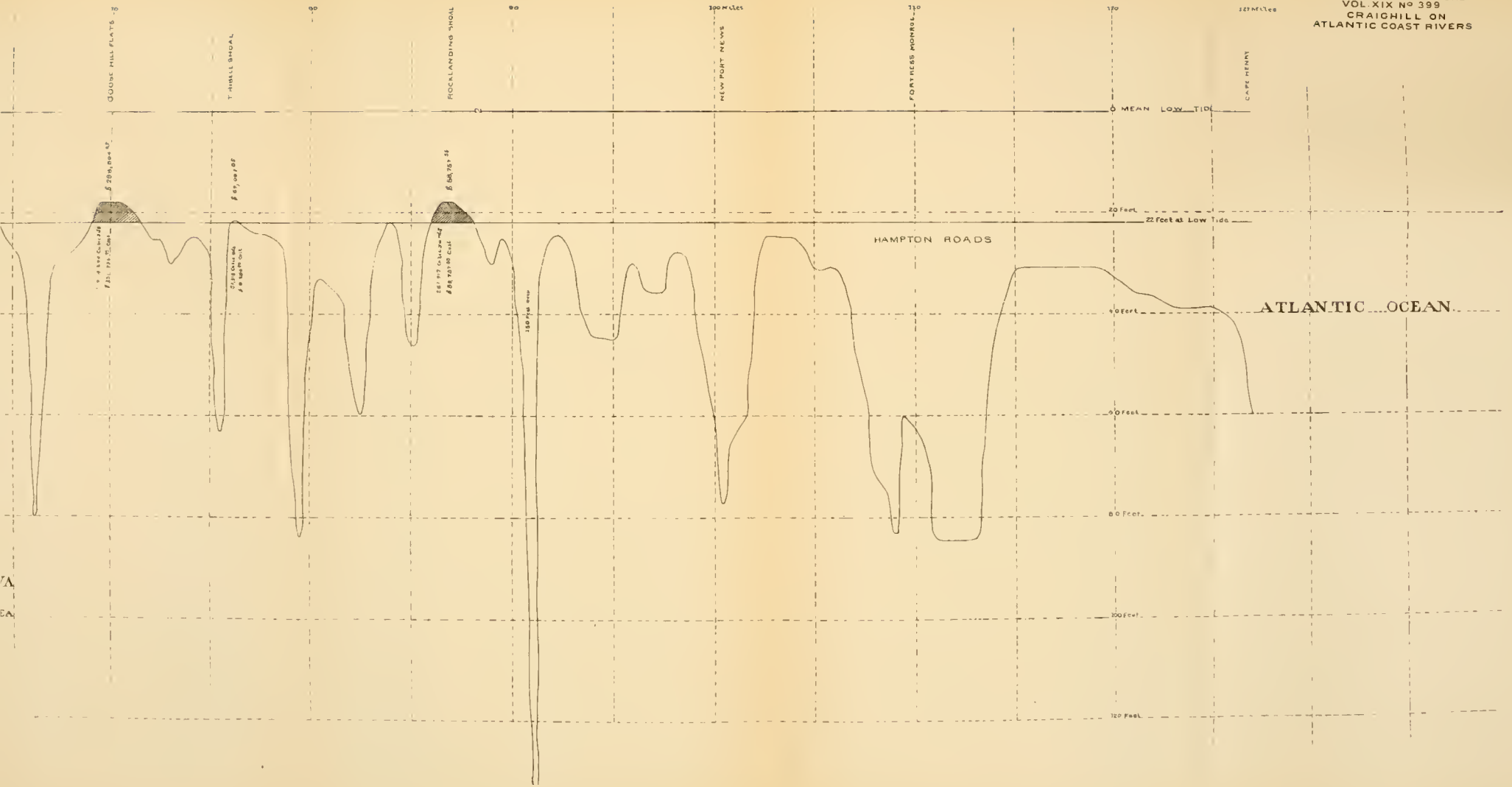
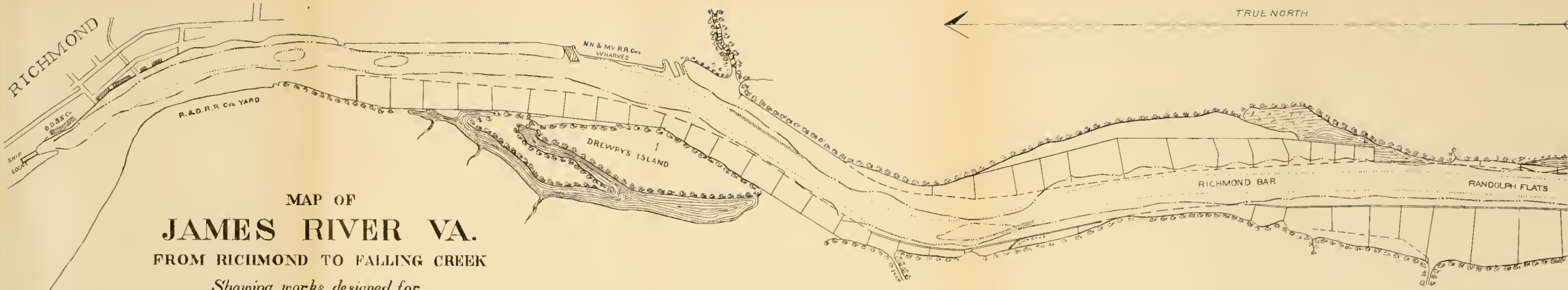

RICHMOND
RICHMOND BAR
DREWRY'S BLUFF
CHAFFIN'S BLUFF
KINGSLAND REACH
DUTCH GAP
VARINA
DEEP BOTTOM
CURLE'S NECK
COX'S BAR
BERMUDA HUNDREDS
CITY POINT BAR
HARRISON'S BAR
FORT POWHATAN
DANCING POINT
SWAN POINT
0 Feet MEAN LOW TIDE
22 Feet at Low Tide
MEAN RISE & FALL TIDE at Richmond = 3.5 Feet
" " " " SWAN POINT = 2.0 "
" " " " Fortress Monroe = 2.6 "
CHANNEL
of
JAMES RIVER VA
from
RICHMOND to the SEA
1885

PLATE XXXVI
TRANS. AM SOC. CIV. ENGRS
VOL. XIX Nº 399
CRAIGHILL ON
ATLANTIC COAST RIVERS





MAP OF
JAMES RIVER VA.
FROM RICHMOND TO FALLING CREEK

Showing works designed for its improvement.

SCALE

1000' 0' 1000' 2000' 3000' 4000'

1887.

Note.

Mean rise and full of tide = 3.5 ft.

Longitudinal dyke proposed shown thus. ----

" " *constructed* " " ═══

Wing-dams " " ——

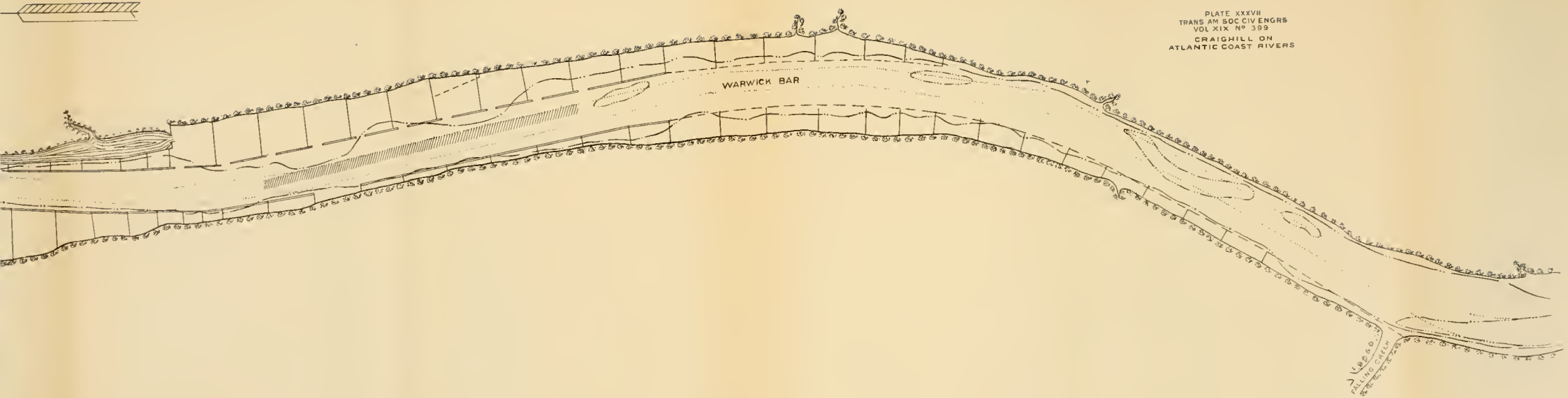
Lines of proposed channel " "

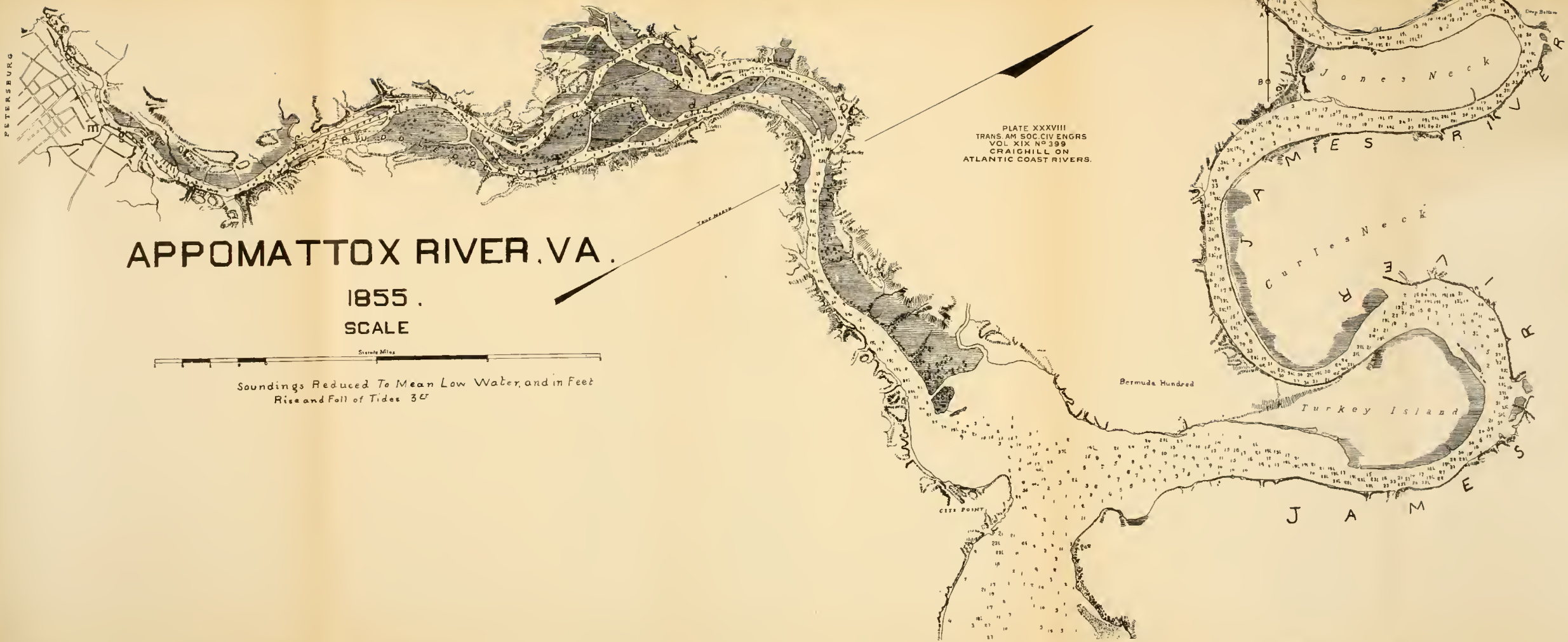
Area dredged in 1887 " "

2 fathom contour " " —·—

3 " " " " —··—

PLATE XXXVII
TRANS AM SOC CIV ENGRS
VOL XIX Nº 399
CRAIGHILL ON
ATLANTIC COAST RIVERS




PETERSBURG
APPOMATTOX RIVER, VA.
1855.
SCALE
Statute Miles
Soundings Reduced To Mean Low Water, and in Feet
Rise and Fall of Tides 3ft
PLATE XXXVIII
TRANS. AM. SOC. CIV. ENGRS
VOL. XIX. No. 399
CRAIGHILL ON
ATLANTIC COAST RIVERS.
Jones Neck
JAMES RIVER
Curles Neck
Bermuda Hundred
Turkey Island
JAMES
CITY POINT

POCOMOKE RIVER
SCALE
Original Survey Made In 1878
Original Curve of 7 FT Shown Thus
Original " " 5 FT " "
Improvements Made between 1878-80
" in Progress 1888
Plane of Reference Mean Low Water
Rise and Fall of Tides 3.0
PARTNERSHIP ISLAND
a
b
c
d
e
f
g
h

R MD.

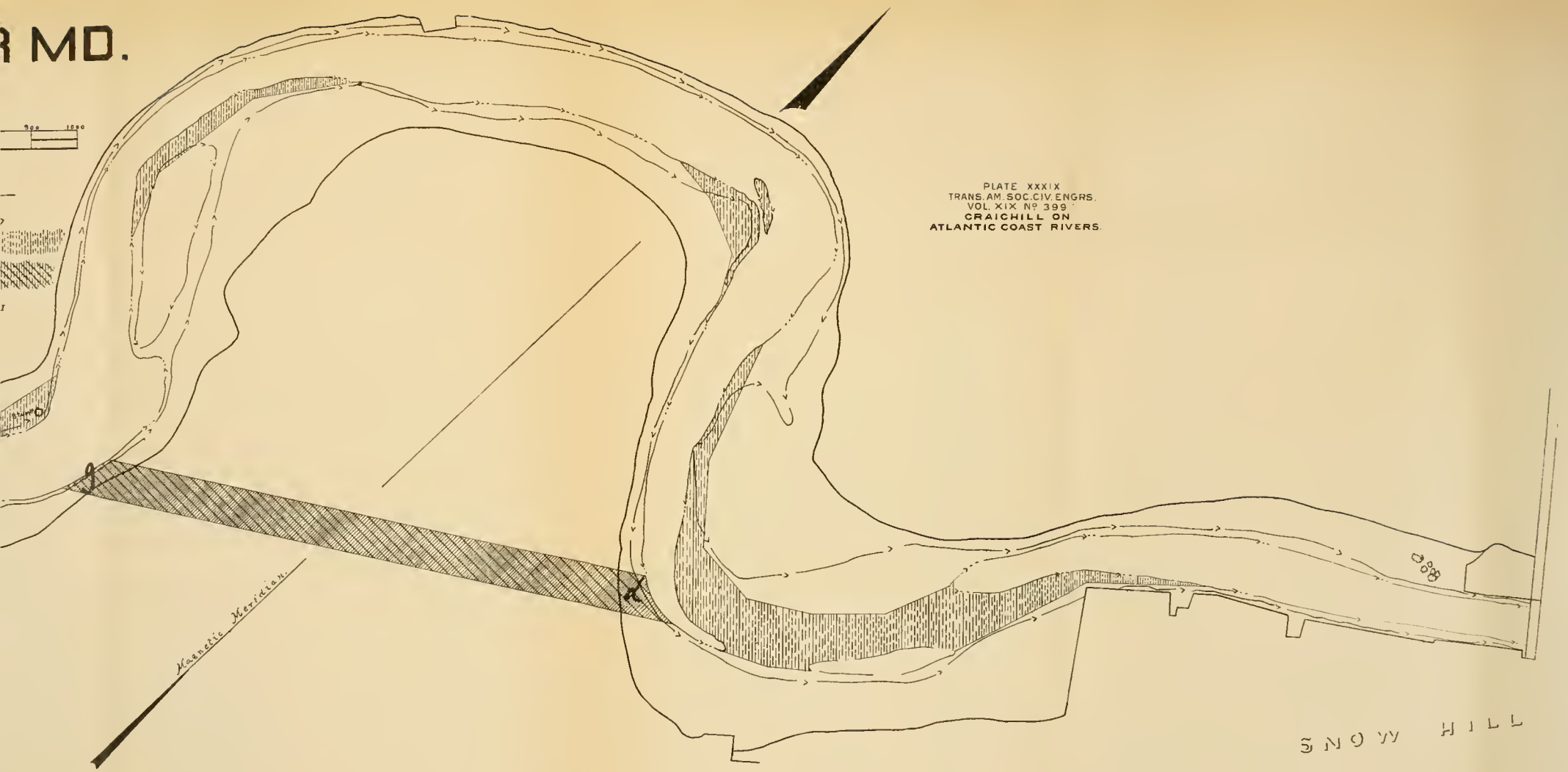


PLATE XXXIX
TRANS. AM. SOC. CIV. ENGRS.
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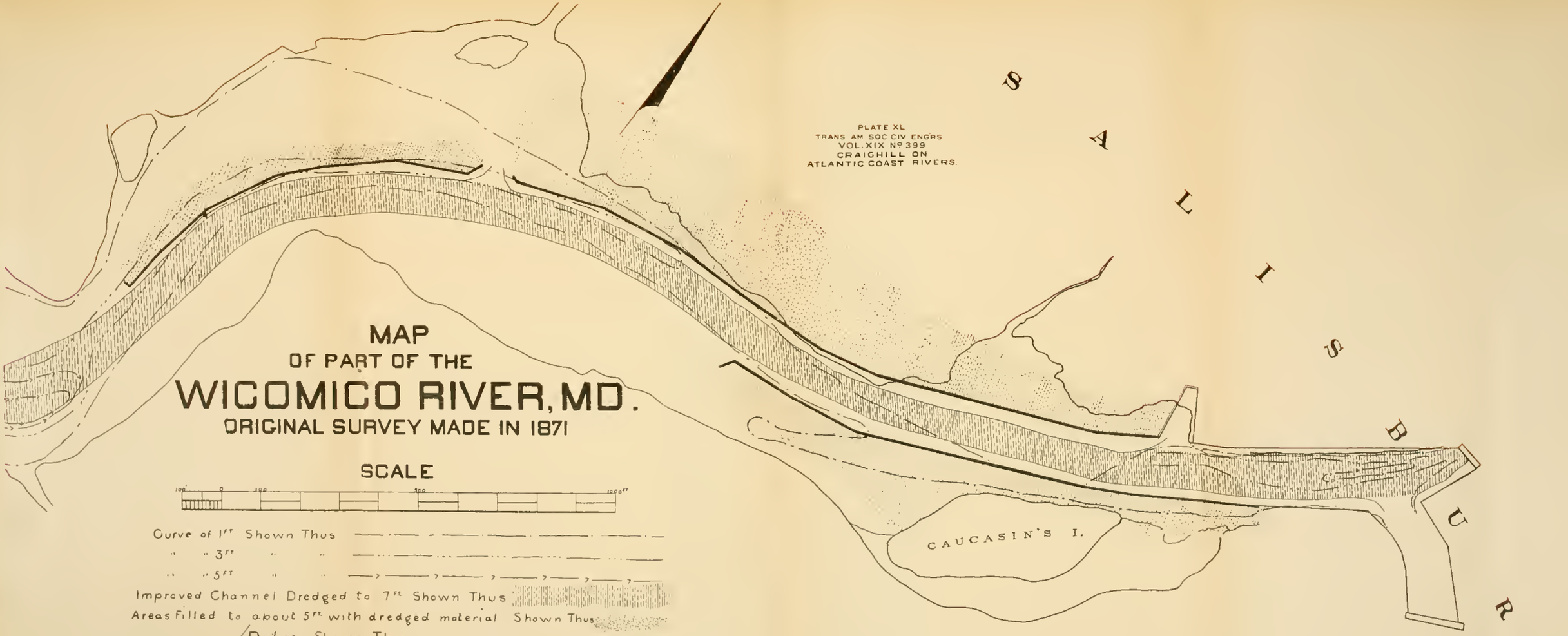
CRAICHILL ON
ATLANTIC COAST RIVERS.

PLATE XL
TRANS AM SOC CIV ENGRS
VOL. XIX Nº 399
CRAIGHILL ON
ATLANTIC COAST RIVERS.



MAP
OF PART OF THE
WICOMICO RIVER, MD.
ORIGINAL SURVEY MADE IN 1871

SCALE

100 0 100 200 1000 ft

Curve of 1ft Shown Thus
" " 3ft " "
" " 5ft " "
Improved Channel Dredged to 7ft Shown Thus
Areas Filled to about 5ft with dredged material Shown Thus
Dykes Shown Thus
Plane of Reference Mean Low Water
Rise and Fall of Tides 3ft.

DISCUSSION.

F. Collingwood, M. Am. Soc. C. E.—The paper so unostentatiously put forward by Colonel Craighill is one of general interest to the profession. An engineer never knows how soon he may be called upon to do something in the training and control of rivers, and, therefore, every one should possess at least a general knowledge of this important subject. The several examples described in the paper seem to have been successfully dealt with, and as success is the proof that proper methods have been pursued, they become examples for study and not for criticism. Intelligent criticism, too, is hardly practicable without more of detail than is given.

The leading idea in these works seems to have been the true one, viz., to work with nature, thus economizing effort, and obtaining permanency in the established channels. The fact, for instance, that at Baltimore there was a line on which the flow of the rivers concentrated, where the bottom was at many points too hard for the stream to act upon, all sediment being swept away, led to the conclusion that if the artificial channel were removed to this line, natural forces would keep it open. This reasoning was fully sustained by subsequent results.

So far as I have seen the rivers of Virginia, their tide water reaches are mostly through light, cavy soils, so that the banks are not permanent. The James River, for example, has a width at Newport News of about five miles, of which a wide reach on each side is made up of shifting sands, the water deepening slowly to 9 to 15 feet at the edge of the channel, and then sharply to about 75 feet in the channel. Every year the bluffs cave off more or less, and the material is carried along by wind and wave, requiring frequent dredging to keep a sufficient depth of water along piers and bulkheads. This wear is plainly shown at Jamestown, further up the river, where an earthwork thrown up during the war, in front of the old historic church, has had the front embankment entirely cut away. In traversing an old survey made some fifty years ago, two miles above Newport News, I found some 40 feet had been cut off across the river front.

This being the case, it is evident that the methods pursued in improving the channels must be similar in character to those adopted on the Mississippi; that is, not only to train the stream, but to protect the banks from wash. In the wide reaches the waves are the most active agents of destruction, but further up, where the stream narrows greatly, the currents, doubtless, act most powerfully.

As no details of currents, discharge, or construction of works are given, it is useless to attempt any discussion of exact methods.

The Dutch Gap cut-off is spoken of with approval. The question of permanency cannot be dealt with without fuller knowledge of the conditions of the river at this point.

As to the question of the "tentative" treatment of rivers, it will be remembered that Captain Eads (cited by Colonel Craighill) always held to a very positive treatment. In other words, having made careful observation of all the elements of the problem in hand, he would advocate a bold and radical treatment, as being most likely to lead to success.

As the writer of the paper justly points out, however, the opposite course is almost necessarily pursued by Government engineers, for the simple reason that appropriations are made by piecemeal; and it is impossible to do otherwise than attack the most salient features of a given problem, in the hope that by reaching quickly a measure of success, further appropriations may be obtained. It should be added that no person was more ready to make changes in plans than Captain Eads, should the necessity become apparent in the progress of work.

The remarks about the ridicule and abuse heretofore cast upon some of the schemes for improvement of rivers and harbors are fully justified.* If European governments find it to the general interest of the community to improve rivers and multiply canals in every direction as a means of furnishing cheap channels of transportation to regions otherwise isolated from the trade of the world, we can surely make no mistake in following their example. By our present halting policy we make all such works cost vastly more than they ought. Instead of letting them depend on the cupidity or self-interest of politicians, a system should be devised by means of which the initiation of a work should depend on its importance and urgency, and its progress and management be active and business-like. Having once been decided upon, appropriations for the entire work should be made, and its construction and management should be independent of political intrigue.

C. P. E. Burgwyn, M. Am. Soc. C. E.—In the outset of my contribution to the discussion of the article written by Colonel Craighill on the subject of "The Improvement of Several of the Rivers of the Atlantic Coast," it occurs to me to say that one of the most valuable of the "observations" is that of comparing the treatment of a river by an engineer to the treatment of a patient by a physician.

It is doubtful if a happier analogy could be supplied, and while the brevity of the article forbade going too much into detail, yet the simile could with great propriety be carried still further. For instance, just as a patient needs one kind of treatment for affections of the head, another for the heart, and another for the limbs, the whole treatment bearing a specific relation to the body; so the treatment of one part of a

* For a description of the great changes wrought in the channel of the Clyde, and the immense trade resulting, reference is made to London Engineering of July 27, 1888, for abstracts from a paper before the Institution of Naval Architects by Mr. James Deas.

river should vary from the treatment of a different part, and what would be necessary or proper for one locality might be entirely erroneous for another. Careful study must be undergone, the diagnosis must first be made before the remedy can be applied; but with the advantage to the physician that his mistakes can be concealed by the grave.

One great trouble with the improvement of a river, as Colonel Craighill wisely suggests, is that those in charge of it very often resort to text-books and other sources of theoretical information without studying the river itself. Of course, a reason for this is found in the fact that it has only been within the last twenty years that our rivers and harbors have been studied scientifically and worked upon systematically; and while to the speculating railroad investor, who wishes to see his money turn over at once, twenty years may seem a long time, yet as a matter of fact it is but a short limit for observations to be extended over in order that *all* the physical characteristics may be had as data. Insufficiency of data is as bad as inaccuracy in data. Thus works might be designed to carry all the water in a stream according to information of late times, yet an extraordinary flood might occur, which would destroy not only these works, but, retarded by them, might overflow and ruin regions never overflowed before. Such a flood actually did occur on the James (one of the rivers mentioned by Colonel Craighill) in 1870, when the water rose to 27 feet above low tide—a height ten feet greater than had ever been known positively before; although there was a discredited legend that 100 years ago a flood had reached that great height. Twice since 1870 have these great inundations occurred. Had their re-appearance not been provided for, disastrous results would have followed the effort to control the river. Another instance of the necessity of long study of a river is evidenced by an inspection of the following table:

CROSS-SECTION OF JAMES RIVER AT RICHMOND BAR, FROM JETTY 30 TO JETTY 40.

Date.	Width of River.	Cross-section Area. Low tide.	Greatest Depth.	Average Depth.
	Feet.	Square Feet.	Feet.	Feet.
1874	1 200	6 008	9.2	4.97
March, 1878	590	5 840	11.9	9.90
September, 1878	500	5 544	13.1	11.09
December, 1878	480	5 443	13.2	11.34
March, 1881	470	5 856	15.2	12.46
May, 1882	450	5 776	16.1	12.84
April, 1884	445	5 911	16.3	13.25
June, 1885	445	5 968	16.3	13.41
March, 1887	445	6 028	16.4	13.55
August, 1888	445	6 026	16.4	13.55

It will be seen that in 1874 the river had a width of 1 200 feet, a cross-section of 6 008 square feet, and an average depth of about 5 feet. The width was successively reduced six different times in a period of four years. During this epoch great freshets occurred, large ice flows and various other physical influences were brought to bear, yet in August, 1888, with a width nearly one-third less, the cross-section was three-tenths of one per cent. of what it was in 1874, while there was a gain of over 7 feet in the mean depth. Nothing but a study extended over a long period of time could accumulate these data. The cross-sections were taken by stretching a steel wire across the river, marked at every 10 feet, and sounding with a pole graduated to feet and tenths. The data thus obtained can be considered accurate.

Having collected these facts, the next question is to know whether this same law of constancy of low water areas would hold at different points on the river; how it would be modified by the gradually increasing form of section, which a tidal river must have as you approach its mouth; what would be the effects of curvature, and what should be the curve of the cross-section? Again, how must the section be constructed so as to admit of the discharge of the increased volume of water, when a great freshet is inundating a country, without allowing the velocity to increase to a dangerous degree? Evidently there is much that must and can only be learned by experience.

The great problem of the effect of a cut-off on a tidal river has been carefully studied on the James, and worked out with eminent practical success at Dutch Gap (Plate XXXV). The length of the cut is only 160 yards, yet it saves a long and tedious circuit of 5½ miles, and avoids shoals that had been a great drawback to navigation from the very beginning. When the cut was first undertaken, careful observations were made so as to eliminate all local action, for the mean low water plane was a difficult thing to determine. The area of the cut-off was sounded out, so as to make a future comparison, and as each successive enlargement was made, the same observations were gone over. Finally, after the gap had been worked upon at various intervals for about fifteen years, the entire course of the river has been permanently changed, and the old channel is slowly filling up. Over a million cubic yards have been deposited in the basin. The river goes through an arc of about 180 degrees with a diameter of 1 440 feet.

The effects of making the cut on the height of tide were predicted by the river pilots as about to be most disastrous. They argued it would be like a ditch, that the river above would be drained to a great extent, and that the tides would no longer rise to the usual heights. But, possessed of the accurate knowledge that the various surveys had given him, Colonel Craighill executed the work without regard to ignorant opinion, and its completion justified his confidence in these researches.

As a matter of fact, the cut-off has had a slight effect in raising the

high tide mark at the head of tide-water, and a slight effect in lowering the low tide mark. That is, the range of tide has been increased, but only, however, by a small amount. The "luni-tidal interval" is shorter, while the duration of "stand" is somewhat longer. The great feature of the cut-off, outside of the saving in distance, is the facility it affords for discharging water in a freshet. The usual slope of the river being about 1 foot to the mile in a freshet, it follows that at the Gap there is an amelioration in the slope of from 3 to 4 feet. This benefit is almost inestimable, for sometimes an extra foot of rise means the destruction of a large amount of property. The deposit of a million cubic yards is a serious matter. It will not do to attempt a change in the forces of nature without thorough knowledge. As Colonel Craighill most truly says: "One ounce of experience is worth tons of theory."

When the work of the improvement of the river was commenced, the navigable depth was only a little over 10 feet at high tide. At the present time (September, 1888) it is a little over 17 feet. Under Colonel Craighill's treatment there has been a distinct and positive gain of 7 feet in the practical draft of such vessels as now come to the port of Richmond. Applying the rule that the value of a port varies as the cube of the water depth, and taking 10 feet as the depth when the work was commenced, and 17 feet as the present depth, the value of the improvement will be as 10^3 to 17^3, or about as 1 to 5. To test this value in another way, careful tables were made of the rates on water freights before the improvement was commenced and at the present time. Now, by multiplying this difference in freight rate by the total quantity of freight carried, it has been found out that this product represents the enormous saving of a million and a quarter dollars annually.

Lieutenant O. M. Carter, M. Am. Soc. C. E.—The Savannah River affords an excellent illustration of the importance of the correct regulation of the height of contracting spurs mentioned by Colonel Craighill.

In 1885 two spurs or wing dams, springing from opposite banks of that river, were built for the improvement of a shoal known as the Long Island crossing, over which there was, in the unimproved condition of the river, a low-water depth of 12 feet. Within three months after the completion of the wing dams a 15-foot mean low-water channel, not less than 300 feet in width, was scoured out.

In August of that same year the dams were injured by a cyclone and their crests lowered to an average level of about mean low water. Shoaling immediately began to take place, and, there being no funds available for repair, the river soon reverted to about its original condition.

In 1886 both dams were repaired. The desired mean low-water depth of 15 feet was again obtained by scour, and has since been maintained by the dams.

The fact that the outer part of a wing dam or spur is what mainly does the work of scour, has been taken advantage of upon that river to reduce the length of spurs, in some suitable cases of excessive channel width, by constructing detached spurs. These have been successful, and no scour has taken place between their inner ends and the shore.

When the proper channel width is determined by building spurs and subsequently connecting their channel ends by training walls, as suggested by Colonel Craighill, a far lighter and cheaper method of construction may often be used for the latter than would be possible if no spurs were built.

Although the system of works most suitable for the regulation of any tidal river can be determined only by a study of that particular river, I think that in general upon tidal streams having soft, easily eroded bottoms, training walls, or a combination of training walls and spurs, give better results than spurs alone. The currents impinging against the ends of the spurs scour out pockets of excessive depths there, while between the sets of dams are invariably found shoals of greater or less extent, the river bed being thus very irregular, and ill adapted to the rapid transmission of the tidal wave and to the filling of the tidal basin in the upper reaches of the river.

Supplementary remarks to paper of Colonel W. P. Craighill on Improvement of the Rivers of the Atlantic Coast.

Captain William H. Bixby, M. Am. Soc. C. E.—The remarks of Colonel Craighill interest me very greatly, especially because they concern streams similar to those upon which I have been busied for the last four years. The experience of these years leads me to confirm most, if not all, of Colonel Craighill's remarks.

In the regularization of such streams I have found the same necessities for treating each according to its own peculiarities. The best examples of regularized streams are undoubtedly to be found in Europe, but European practice cannot be strictly followed in the United States because of many reasons. In Europe, time, money and private authority or license are of less consequence, and perfection of work is of greater consequence, than in the United States. In my opinion the best engineering is not always that which gives the most perfect results; but the best engineering is rather that which gives the best and most useful results per dollar of expenditure. Had all the money ($47 000) voted by Congress from 1880 to 1887, for the improvement of the Great Pedee River, S. C., been spent in doing perfect work from the mouth of the river upwards as far as the funds would have allowed, the river might have possibly been thoroughly cleared over a third of its distance, and a commerce of perhaps $400 000 per year might have been thereby developed; but by doing rough work over the full length of the river, a commerce of $2 000 000 has actually been brought into existence. Such cases as this are valid arguments for oftentimes doing rapid, though rough and imperfect, work, even though this work may have to be all done over again at some later period. Such work is often the determining element in the development of a colonization and commerce that nothing will ever stop. The necessity for giving to the United States the worth of its money is merely the statement in other words of the idea of advantageous and economical engineering which Congress requires from its engineers. It is this kind of engineering which suggests the use of cheap materials in such portions of regulating works as will be immediately silted up under the action of river currents; which gives rise to much of the so-called tentative engineering; which puts work in roughly at first, to be finished more leisurely at some later moment; which does work only at the places where it is most immediately needed; which allows one bank of a river to take care of itself, not because it would not be benefited by assistance, but because other places need help more than it; and which even uses a dredge to-day although tolerably certain that the dredging may be all filled up again at some later period, though not before navigation has been benefited in proportion to the money expended.

Another point, which may be deduced from Colonel Craighill's re-

marks and which has been fully confirmed by general experience, is that rivers are generally very difficult to drive while often very easy to lead. In other words, it is very important to examine carefully the individual characteristics and local peculiarities of each river before commencing to prescribe its regularization works, in order to know where the river would naturally go if unhampered by obstructions, and in order to make the regularization works conform as much as possible to the natural route and circumstances of the river currents. Where leading is possible, it is generally far more economical to lead; but where leading is impossible, and driving is necessary, the force must be applied with steadiness and power.

I find that the people living on the rivers of the South Atlantic are always very desirous of making short cut-offs across the natural loops of these rivers without having for a moment considered the natural results of such cut-offs, in the increased velocity at the cut-off and the diminution of depths above the cut-off, which usually accompany the opening of such cuts upon up-country steep-grade streams. Moreover, most cut-offs turn out to be more expensive work than at first estimated. The objections to cut-offs naturally decrease with the grade of the river-bed and the approach to tidal and flat neighborhoods.

As Colonel Craighill states, many people are inclined to throw ridicule upon the improvement of streams which bear the name of "creeks." They forget that the greater portion of the South Atlantic creeks would be called rivers in the Northern, Eastern and Middle States and in Europe. After going up some of these creeks on freight steamboats for forty or more miles, and after having found several hundred thousand dollars' worth of goods shipped annually over them, I have gained a new respect for the term "creek," and have realized that all does not lie in a mere name.

Colonel Craighill states that it had been his wish to add a brief account of the improvement of the Cape Fear River, but that want of time and other causes had prevented.

As I have been in charge of this work for a few years as Colonel Craighill's successor, I feel qualified to speak upon the improvement of this river, and I do so with much pleasure.

The two branches of the Cape Fear River meet at Wilmington. They bring down to Wilmington a fresh-water discharge of about 500 cubic feet per second on an average during the three low-water months, and about 18 500 cubic feet per second during the nine flush-water months, or about 14 000 cubic feet per second on an average throughout the entire year. At Wilmington the river has a cross-section of about 14 000 square feet, so that the height of its water surface at this point remains approximately unaffected by the variations in the amount of the fresh-water discharge. Both branches are subject to tidal influence for a distance of about 30 miles above Wilmington.

From Wilmington to the ocean, 26 miles, the Cape Fear River is a tidal basin of about 37 square miles area, in which the tide rises and falls about 2.5 feet at Wilmington and 4.5 feet at the ocean, or about 3.5 feet on an average. At the river's mouth, 1.25 miles wide, the amount of water which must pass to and fro each tide is therefore about 160 000 cubic feet per second. The ocean bar is about two miles to the seaward of the throat of the river's mouth. (See Plate XLI.)

The improvement of the Cape Fear River below Wilmington has been under the consideration either of the State of North Carolina or of the United States since 1821.

Prior to 1761 the Cape Fear River was reported as allowing vessels of 14 feet draught to pass over its main bar at low water and to enter the river by the same route as they follow to-day. In 1761, however, during a violent equinoctial storm of four days' continuance, the wind and sea made a breach, reported at first as 18 feet deep at high water, and later as not over 2.5 feet deep at low water, and nearly half a mile wide, through the sand banks into the Cape Fear River about 8 miles above its former bar entrance, at a place called Haulover. This breach, cutting off 6 miles of the river's length, gradually increased in importance so as to form a new mouth to the river, deepening from 6 feet at low water in 1797 to 10 feet at low water in 1839, and received the name of New Inlet. The effect of this new mouth upon the river was to diminish the depth of water upon the main bar entrance from 15 feet in 1797 to 9 feet in 1839.

The past and present condition of these bars is shown in the following table, the depths being expressed in feet and being (according to the best attainable information) reduced to low water.

Name of Author of Map.	Date.	Main Outlet.		New Inlet.
		Baldhead Channel.	Rip or Bulkhead or Shoalest part of Western Channel.	
Edward Moseley	1733	14	3	Closed.
William Nimble	1738	16	4	Closed.
Joshua Potts	1797	15	3	6
Colonel Kearney, Topographical Engineers.	1820	10.5	3	6.5
Captain Glynn, United States Navy	1839	9 ?	9	10
Pilots and others	1850	8	9.5	7
Coast Survey, Captain Maffit	1851	8	7	8
" "	1852	7.5	7	8
"	1866	8	8	7.5
"	1871	6.5	8	8
"	1872	8	8	8
"	1877	9.6	7.5	8
United States Engineers	1883	14	7.5	Closed.
" "	1885	14	7.5	Closed.

Prior to the opening of New Inlet, and even until 1839, Baldhead Channel was the natural and main entrance to the river. From 1839

to 1872 both the Rip and New Inlet were the main entrances, and the use of Baldhead was discontinued. Since 1872, and the closure of the New Inlet, Baldhead has again become the main channel, and has been gradually regaining its former depths.

In 1821, the date of the first detailed survey of the river, vessels of over 10 feet draught were obliged to lighter their cargoes 14 miles below Wilmington, 10 feet draught at high water, and 7.5 feet draught at low water, being all that could be carried from the point of lighterage up to Wilmington.

From 1821 up to 1829 the improvement was under the charge of the State of North Carolina. From 1829 up to the present time the improvement has been under the United States.

Between 1822 and 1829, the State of North Carolina, under the guidance of Mr. Hamilton Fulton, State Engineer, attempted to improve the channel between Wilmington and New Inlet by dike closures of the minor channels, by jetty contraction of the main channel-way, and also by dredging across the shoals. A small amount of money (supposed to have been from $60 000 to $80 000) was spent in this way, with moderately good results. This work deepened slightly the water in the main channel and was of especial value as being the commencement of an important river improvement.

By the dikes and jetties thus built, the river currents were restricted in general to a single route; and such scouring as thereafter occurred resulted in a permanent gain in channel depth. One of these dikes, however, that between Big (or Campbell's) Island and the mainland on its west, was expected to force the current to scour a channel east of that island; but this scouring took place very slowly, and the expected results were not finally attained, because of the existence in the desired channel-way of a large quantity of apparently unsuspected logs and stumps. These logs and stumps have had to be removed since at great expense.

In 1826–27 the United States Engineers made a careful survey of this portion of the river.

In 1829, when the United States took entire charge, there were several shoals with a least depth of 7.5 feet at low water between New Inlet and Wilmington.

From 1829 to 1847, by a continuation of the dredging and jettying, in general accord with former work, the depths on the shoals were increased by about 2 feet, so as to allow 9.5 feet at low water, a depth equal to that at the bar entrances.

By 1874 it was evident that the depth on the bar would soon be increased; and therefore under the direction of Colonel Craighill plans were made to deepen the river channels proportionately. The projects of Colonel Craighill of 1874–81 as continued to date proposed: 1st, to locate channels in such positions that the natural flow of the river cur-

rents would thereafter tend continually to keep these channels scoured to full depth; and, 2d, to obtain these channels by dredging and the removal of the before-mentioned logs and stumps. This work has been continued from 1874 up to the present year as fast as possible under the inadequate yearly appropriations of Congress. The results have been eminently successful; so much so that to-day there exists a continuous channel of 16 feet depth at low water over the whole 20 miles above New Inlet. The latter depth, combined with the average rise of tide on the bar (4.5 feet) and at Wilmington (2.5 feet), gives to-day a good 18-foot navigation at high water from the ocean to Wilmington. These channels will probably be completed to 270 feet width and full depth under the present appropriations; and it to-day appears probable that these channels will then remain permanent without the aid of further diking, jettying or dredging. In almost all cases these channels were located along the thread of the supposed natural current.

The lighterage, formerly a necessity over 14 miles of the river, is now entirely given up, and all vessels that can cross the outer bar proceed directly to Wilmington to unload. So far as possible all the work of improvement has been done by contract.

In several parts of the river below Wilmington, at places 2 000 feet from the shore and where there is now 14 feet depth of water, stumps of trees, pine as well as cypress, have been found firmly imbedded in the river bottom in an erect position, apparently surrounded by the soil in which they originally grew. Several theories have been advanced to account for these strange facts, such as that the ground has subsided, or has been undermined since the death of the trees; but none of these theories are entirely satisfactory. Many of these stumps had to be removed; and in their removal the clam-shell dredge developed a remarkable efficiency for such work, rendering the use of explosives almost unnecessary.

In 1829, when the United States Government assumed active charge of the improvements, the river had three bar entrances with least depths about as follows: 9 feet at the Baldhead Channel, 9 feet at the Rip Channel (both near the old river mouth), and 10 feet at the New Inlet Channel; these bars being, respectively, 9, 6 and 2 miles below the point which was then the head of the river's delta. No appropriations were made for work at the river's mouth until 1852, when Congress voted $160 000 for this purpose. Under these appropriations it was proposed to build a dike, 4 400 feet long, across the openings in Zeke's and Smith's Islands on the south side of New Inlet, so as to stop this inlet from growing larger and to check the outflow of water by this route; and it was also proposed to build a jetty at Baldhead Point, to check further widening of the Cape Fear main outlet. All this work was completed as far as the funds allowed.

The jetty, 300 feet long, constructed by Captain Woodbury in 1853

at Baldhead Point, has remained in tolerably permanent condition up to to-day (1885), although its shore end was flanked by the sea in 1871, and although it now stands under water and nearly 1 500 feet in advance of the present shore-line. Between the jetty and the shore the water still remains quite shoal. This jetty was originally built to protect Baldhead Point against further and rapid erosion by the waves and currents. It appeared to accomplish that object until 1870, the date of commencement of work at New Inlet. Its effect, if any, upon Baldhead and the Rip Channel, cannot be stated with certainty.

The closure from Zeke's Island eastward to Smith's Island was attempted by means of a continuous line of very heavy stone riprap, efforts being made to assist the rapidity of the construction by the temporary use of auxiliary perpendicular jetties or piles, sheet-piles, brush, and sand-bags. When it was nearly completed, the specially violent storms of 1857 destroyed the work beyond repair by the funds then available. Subsequent experience in work of similar nature at the same locality leads to the belief that this failure was due, not to the method, but only to the small amount of stone allowable by the moneys then available.

The work of 1852–57 around New Inlet was tentative, and though not permanently successful, was valuable as indicating a solution suggested by the Board of 1853 and recommended by that of 1858, viz., the closing of the minor and accidental outlet, and the concentration of the main volume of the river outflow into the original and natural main outlet.

When further appropriations became available in 1870, and under the direction of Colonel Craighill, in 1870 to 1873 it was decided to completely close New Inlet, to reconstruct the demolished dike along the south side of the Inlet and thus force the entire river discharge through its old route *via* Baldhead channel; and it was decided also to assist the scour on Baldhead Bar by dredging.

The reconstruction of the dike on the south side of New Inlet, from Zeke's Island eastward to the north end of Smith's Island, was made of a continuous line of wooden cribs loaded with smaller stone. The cribs with vertical sides were used from motives of economy to avoid the necessity of excessive side slopes of stone-work, timber being cheap and stone expensive in this locality. The entire dike closure, 4 403 feet in total length, which was not actually commenced until the fall of 1870, was finished in 1873, at an approximate cost of $60 per linear foot, and with great success. The sand has filled against it to such an extent that it is now, for the greater part of its length, several hundred feet distant from the water at high tide. The construction of this dike was a necessary preliminary to the closure of New Inlet.

As another preliminary to the closure of New Inlet, an experimental deflecting jetty was commenced and built a short distance at the point

where the river current turned from the Cape Fear towards New Inlet. This deflector, built of wooden cribs filled with stone, was continued a short distance only, and then it was decided to make this deflector form a portion of the final closing dike across the Inlet.

The experience gained in the construction of the above dikes and deflectors showed the danger of scour (occasionally 12 feet deep) under loaded cribs with solid bottom, and around the advancing ends of dikes in process of construction, especially if the cribs or dikes are raised at first to full height. It showed also the advantage of open bottoms for such cribs. It indicated the method later used in the closure of New Inlet, viz., the use at first of mattress foundations, and low submerged riprap dikes entirely across the place to be closed, to be followed afterwards by a gradual elevation of the entire dike, commencing always at the shore ends. It showed also the ease with which sand banks can be raised several feet by planted sea oats, by catch fences and by brush, and at the same time the necessity for further protecting such artificial banks against subsequent high storm tides and waves.

The work upon the final closure of New Inlet (commenced in 1875 and finished in 1881) deserves especial note, as it resulted in such complete success.

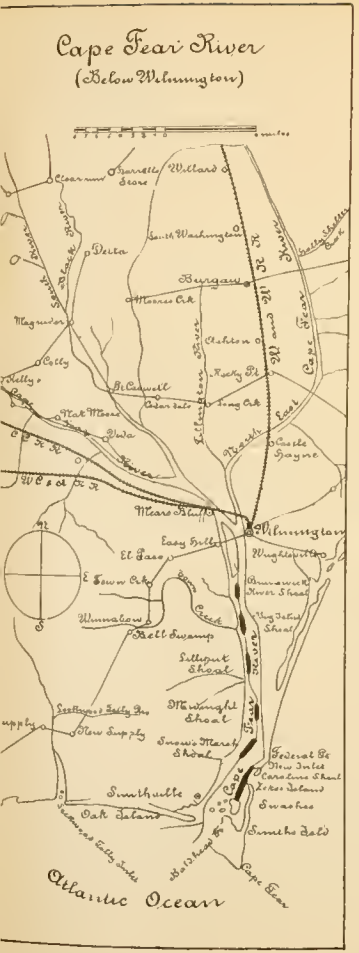
This closure was made of moderately small stone riprap, laid originally on log and brush mattresses, and capped with carefully placed heavy stone (weighing from ½ to 4 tons per piece); the small stone (one-man-stone) being used from motives of economy, and the large, heavy stone being used as a protection against storm waves. The log and brush mattresses were used from a desire to utilize the materials of the country, and because they were one of the elements of cheapness in the bid of the successful contractor. These mattresses were made of pine logs cut in the creek near at hand, formed into rafts of proper dimensions, loaded with as much pine brush as they would carry, towed directly to their places, and sunk by stone thrown upon them from scows. The middle portion of the dam (a length of 3 600 feet) is from 75 to 120 feet broad at its base, and is on an average of 37 feet high ; its side slopes are 1 on 1.5 in the river side, and 1 on 2 in the ocean side. The whole dam is 4 800 feet long, contains about 182 821 cubic yards of material, and cost, complete, about $498 000, or about $105 per linear foot, or $2 75 per cubic yard. At present there is a difference often of 2 feet and sometimes of 3.4 feet between the water levels of its river and sea side. During the construction of the dike the scour around its base, due partly to the river current and partly to the over-fall, was so considerable, and the attacks of the teredo were so severe, that the log and brush foundations were to a great extent undermined or eaten up ; in consequence, continual additions of stone riprap became necessary, so much so that practically the resulting foundations consist to a great extent of small stones. In view of this later experience it would seem that the

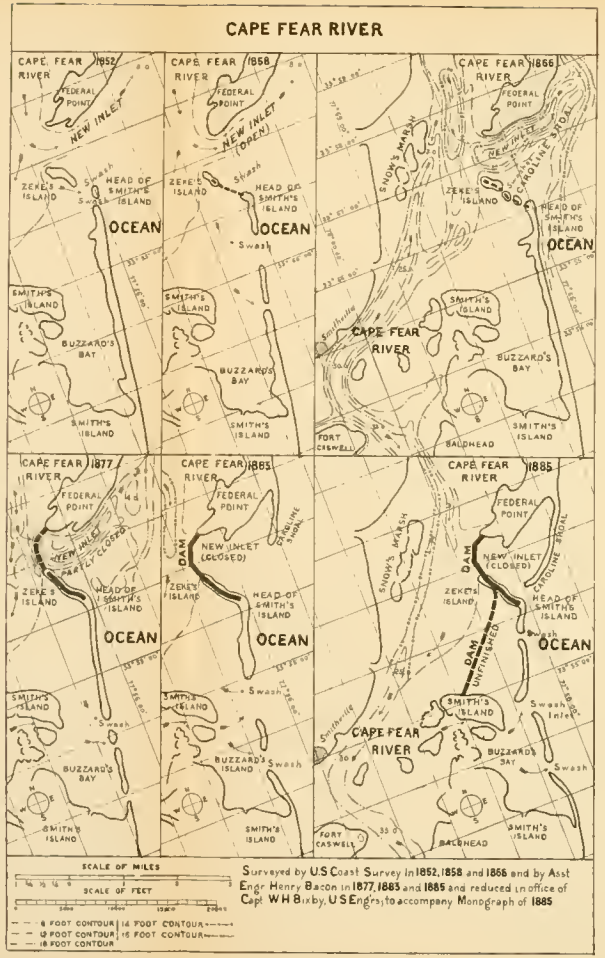
use of small riprap stone foundations from the commencement might have been simpler, quicker, more efficient and cheaper than the use of the log and brush foundations, although the latter were originally suggested by motives of economy.

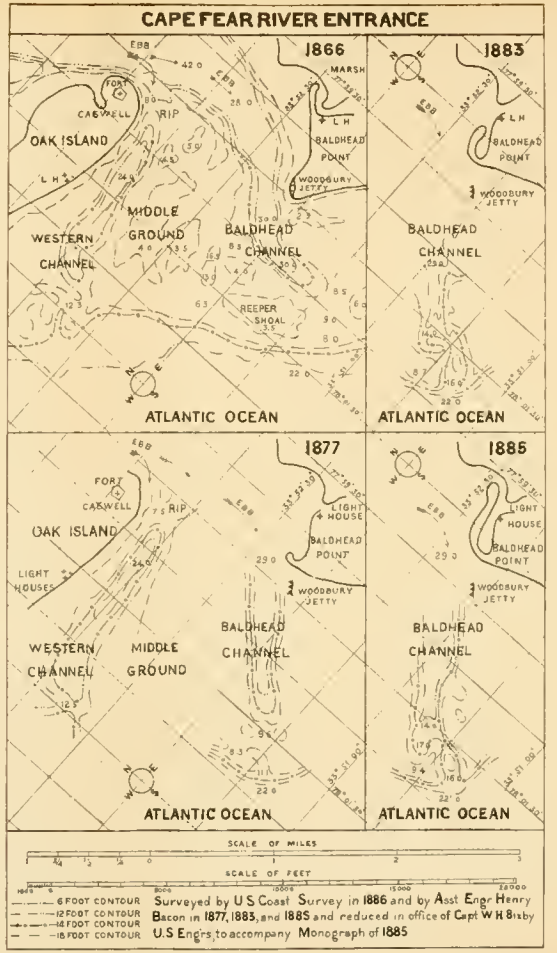
By the closure of New Inlet (combined with the preliminary closure from Zeke's Island eastward to the ocean beach of Smith's Island) the former coast line was re-established; a shoal, 1.5 miles long and several hundred feet broad at high water, gradually built itself up completely across the old bar entrance of New Inlet; and at the mouth of the river the old channel gradually scoured out to 14 feet depth (against 9 feet depth in 1875).

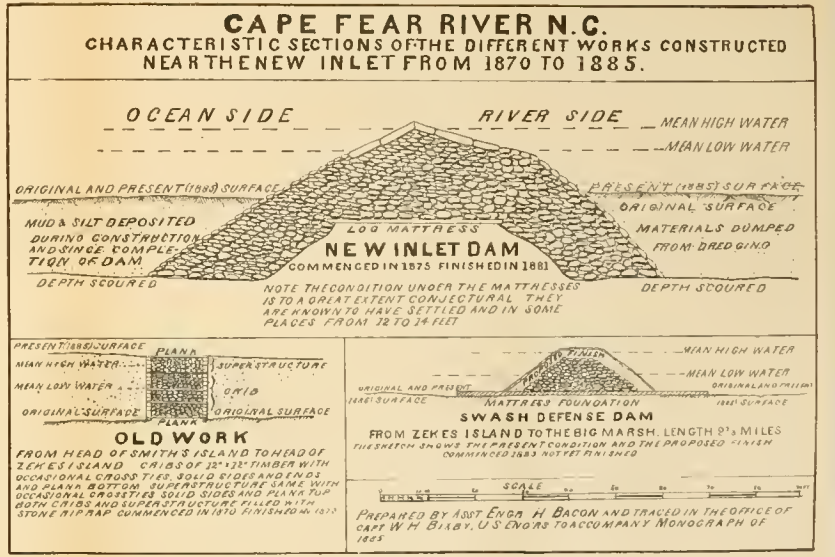
As soon as New Inlet was closed, the river sought to flank this work by crossing over to the ocean south of the closure; and a new dike from Zeke's Island, southward to the south part of Smith's Island, was the natural sequence of the work just described. A first experimental attempt was made, at one of the openings through the strip of sand beach next the ocean, to make the ocean side of Smith's Island continuous by an inexpensive work of round piling, sheet piling, sand bags and timber just in rear of the openings through the beach. A gale arising before the completion of the closure, the width and depth of the opening were so much increased that further attempts to complete the closure in this way were abandoned. The result of this unsuccessful experiment suggested the subsequent construction of a stone riprap dike closure along the neutral line between the ocean and the river, about 1.5 miles in rear of the ocean beach. This new closure, commenced in 1883, and already raised to above low water level, is so far successful, and points to results similar to those that followed the closure of New Inlet. This dike, 11 600 feet long and 7 feet wide on top, rests almost everywhere on a brush mattress foundation, in from 5 to 12 feet of water. The brush mattress was built in sections, each 60 (or less) feet wide, 2 610 (or less) feet long, and 12 inches in final thickness, and cost in place about sixty-six cents per square yard, or $2 65 per cubic yard. The mattress is loaded with 74 000 cubic yards of stone. The dike (foundation included) has cost about $200 000, or less than $18 per linear foot. It rises to the height of high spring tides and has a side slope of 1 on 2. The completion of this new dike (together with the closure of New Inlet) has rendered unnecessary the maintenance of the old dike eastward from Zeke's Island direct to the ocean.

During the progress of the closure of the Inlet, Colonel Craighill decided to hasten and assist the natural scour of the river on the Baldhead Bar. For this purpose, in 1873 and 1874 a steam propeller (Woodbury), of 145 tons burden, was fitted up as a suction dredge (with a double oscillating 10-inch cylinder engine, driving a 9-inch centrifugal pump, and with sand-bins of 40 cubic yards capacity), and proved so successful that it dredged sand in 8 to 14 feet of water upon the bar


Cape Fear River
(Below Wilmington)
Wilmington
Smithville
Southport
Oak Island
Bald Head
Fort Caswell
Cape Fear
Atlantic Ocean


CAPE FEAR RIVER
CAPE FEAR RIVER 1852
CAPE FEAR RIVER 1858
CAPE FEAR RIVER 1866
CAPE FEAR RIVER 1877
CAPE FEAR RIVER 1885
FEDERAL POINT
NEW INLET
NEW INLET (OPEN)
NEW INLET (CLOSED)
ZEKE'S ISLAND
HEAD OF SMITH'S ISLAND
OCEAN
SMITH'S ISLAND
BUZZARD'S BAY
CAPE FEAR RIVER
FORT CASWELL
BALDHEAD
SNOW'S MARSH
CAROLINE SHOAL
DAM
DAM UNFINISHED
SCALE OF MILES
SCALE OF FEET
6 FOOT CONTOUR
12 FOOT CONTOUR
18 FOOT CONTOUR
14 FOOT CONTOUR
15 FOOT CONTOUR
Surveyed by U.S Coast Survey in 1852, 1858 and 1866 and by Asst Engr Henry Bacon in 1877, 1883 and 1885 and reduced in office of Capt W H Bixby, U S Engrs, to accompany Monograph of 1885


CAPE FEAR RIVER ENTRANCE
1866
1883
1877
1885
OAK ISLAND
FORT CASWELL
RIP
MARSH
LIGHT HOUSE
BALDHEAD POINT
WOODBURY JETTY
WESTERN CHANNEL
MIDDLE GROUND
BALDHEAD CHANNEL
REEPER SHOAL
LIGHT HOUSES
ATLANTIC OCEAN
SCALE OF MILES
SCALE OF FEET
6 FOOT CONTOUR
12 FOOT CONTOUR
14 FOOT CONTOUR
15 FOOT CONTOUR
Surveyed by U.S Coast Survey in 1886 and by Asst Engr Henry Bacon in 1877, 1883, and 1885 and reduced in office of Capt W H Bixby U.S Engrs, to accompany Monograph of 1885


CAPE FEAR RIVER N.C.
CHARACTERISTIC SECTIONS OF THE DIFFERENT WORKS CONSTRUCTED NEAR THE NEW INLET FROM 1870 TO 1885.
OCEAN SIDE
RIVER SIDE
MEAN HIGH WATER
MEAN LOW WATER
ORIGINAL AND PRESENT (1883) SURFACE
PRESENT (1885) SURFACE
ORIGINAL SURFACE
MUD & SILT DEPOSITED DURING CONSTRUCTION AND SINCE COMPLETION OF DAM
LOG MATTRESS
NEW INLET DAM
COMMENCED IN 1875 FINISHED IN 1881
MATERIALS DUMPED FROM DREDGING
DEPTH SCOURED
NOTE THE CONDITION UNDER THE MATTRESSES IS TO A GREAT EXTENT CONJECTURAL THEY ARE KNOWN TO HAVE SETTLED AND IN SOME PLACES FROM 12 TO 14 FEET
PLANK
SUPERSTRUCTURE
CRIB
OLD WORK
FROM HEAD OF SMITH'S ISLAND TO HEAD OF ZEKE'S ISLAND CRIBS OF 12" x 12" TIMBER WITH OCCASIONAL CROSS TIES, SOLID SIDES AND ENDS AND PLANK BOTTOM SUPERSTRUCTURE SAME WITH OCCASIONAL CROSSTIES SOLID SIDES AND PLANK TOP BOTH CRIBS AND SUPERSTRUCTURE FILLED WITH STONE RIPRAP COMMENCED IN 1870 FINISHED IN 1873
SWASH DEFENSE DAM
FROM ZEKES ISLAND TO THE BIG MARSH, LENGTH 9⅞ MILES
THE SKETCH SHOWS THE PRESENT CONDITION AND THE PROPOSED SYSTEM
COMMENCED 1883 NOT YET FINISHED
SCALE
PREPARED BY ASST ENGR H BACON AND TRACED IN THE OFFICE OF CAPT W H BIXBY, U S ENGRS TO ACCOMPANY MONOGRAPH OF 1885

entrance in all kinds of weather at a cost of not over sixteen cents per cubic yard. This steamer was afterwards remodeled and its suction dredge apparatus much altered and improved. As altered, it worked more easily and rapidly than before, so as to dredge and pass through its bins as much as 656 cubic yards of sand in one day, at a cost of not over fifteen cents per cubic yard. The usefulness of this dredge, however, is not to be measured merely by the quantity of sand passed through its bins, inasmuch as its drags loosen and stir up from the channel bed a large quantity of material, which is carried directly into the adjacent deep water by the river and ocean currents.

During the progress of all this work, the rapidity and economy of construction were continually hampered by the lack of sufficient appropriations, which rarely, for any individual year, equalled the estimates of the engineer in charge. The lack of sufficient funds, often just at moments when those funds were most needed, has caused to the work a considerable loss of time and money.

William R. Hutton, M. Am. Soc. C. E.—The paper before us, avoiding much detail, is, as its author states, rather a basis for discussion by the Society of the general question presented, than of the particular works described.

It is true that the methods of river improvement are practical rather than theoretical, if by the latter we understand mere speculation. But true theory also refers to principles of general application in any art or science, and if hese principles are derived from observation and experiment by others, they may be quite as valuable as anything that has taken place solely within the limited experience of any one of us. Such principles of river improvement as have been formulated are the results of such experience. Practical experience is invaluable, but it needs the aid of "theory" to yield its best results.

It was most natural that in the first attempt to improve the entrance to Baltimore, Colonel Brewerton should have adopted the shortest line to reach deeper, unobstructed water nearer the eastern shore of Chesapeake Bay, the more so in view of the small appropriations and generally the small amount of river and harbor work at that time attempted by the general Government. The difficulty of maintaining this channel transversely to the currents of the bay was, however, recognized in Colonel Brewerton's day during our civil war.

The remedy was obvious, but it required not only the "stratum of hard common sense," (a pre-eminent quality in our author), to perceive it, but great courage to recommend at that day so radical and expensive a change. The result could not be doubtful. The currents of both ebb and flood were in the direction of the new Craighill channel; there were no ocean waves to disturb the bottom, and the freshet discharge of the Susquehanna follows apparently in great part the eastern shore of the

bay. Under these conditions a dredged channel should be easily maintained without training walls, which would here be a most inappropriate method of improvement. The sides of the cut, indeed, where of much depth act as submerged walls, and induce the greater current in the deeper water between them.

It may be doubted if the deposit of dredged material in the bay not less than a mile from the channel forces any appreciable quantity of water into the new channel or fills any other useful purpose. Its economy, however, is a justification for the practice.

JAMES RIVER.—The relative efficiency of spurs and training walls in the regulation of a water-way is well established. The former have their advantages in cheapness of first cost, facility of modification and quicker results, all powerful factors in our great country, where so much is to be done, and where results are so impatiently awaited. They are more effective and less expensive to maintain in rivers of gentle current.

The utility of the tidal reservoir between the spurs is, however, doubtful. The improvement of a tidal river is best promoted by such works as induce the tide to ascend as far as possible, by removing all obstructions to its advance through a regulated channel, narrowing gradually from its mouth, somewhat in proportion to the quantity of tidal water passing through each successive section. The spaces between spurs disturb the regular advance of the tidal wave, and prevent its ascent both by the eddies around the ends of spurs and by the quantity of water diverted from the channel to fill the "reservoirs" between them. On the Clyde large quantities of valuable land have been taken from this "reservoir," and the land reclaimed by the longitudinal dikes along the lower Seine has sold for more than the cost of the improvement.

All great improvements have been made with training walls, but they require careful location after thorough study of the stream. The Clyde, the Tees, the Seine, the Rhone, the Loire, the Maas, and the Hudson are instances.

On the James the greatest trouble seems to have been with the straight wide reaches of the river, and at points where the channel crosses from one side to the other, generally due to the changes in direction of curvature. In the former cases the difficulty has probably been excessive cost, for although a curved channel is more easily maintained than a straight one, a proper reduction of the width of the low water channel would hardly have failed of the desired effect. The latter case has been reduced to law, "la loi Fargue."

"When a river is to be treated which cannot economically be kept in proper condition by dredging, the evils may be diminished by making the curves and the width of the bed to vary gradually.

"The rules to be followed are based upon the very considerable and permanent success obtained upon the Garonne.

"Experience has shown that the curves must vary gradually, from the maximum curvature of the concave bank at the crown or summit of the curve, to nothing at the point of contrary flexure. The width also must diminish from the summit of the curve, where the bed is widest, to the point of reverse curvature, where it is much reduced." Instances are given where the width at the point of reverse curvature was with marked success reduced to 150 meters, although the width at the summit of the curve above was 200 meters and at the summit of the very sharp curve below 250 meters. "The conditions of success are also increased when the convex curves are made longer than the concave banks. We know that there is a tendency to form deposits along the convex bank. If, therefore, the banks are convex on both sides at the point where the direction changes the current, repelled from both shores, is concentrated in the middle of the stream, and the depth of the channel is increased." The rules apply also to a tidal river, on condition of a progressive increase of width, demanded by the increased discharge in the lower portions. To obtain the best results, the depth must be regulated as well as the width, but this is more necessary in non-tidal rivers with rapid currents.

The regulation of the channel by scour is not performed in very high floods or at low water. The floods of the Missouri are said to fill up at times the entire channel—to re-excavate it as the water falls, in the same or a different place, according to the conditions of the site. There is generally a stage (which has been called the regulating stage) in which the regulating works are able to control and form the bed. To this height should the work be raised. If lower, the water will escape over them and lose a part of its scouring force; if higher, the flood waters will be concentrated upon the low water bed, causing excavation in places, and dams and bars in others.

It is sometimes the case that the dredge alone can be used for forming a permanent bed, but this is possible only when the disturbing causes recur at long intervals. Most of the great works of river improvement have been the joint work of the dredge and training wall; the former has been the more largely used in Great Britain, the latter on the Continent.

The effect of a cut-off is to concentrate in a short distance the slope which before was distributed over the length of the bend. The excess of slope will in time distribute itself upon the river bed for some distance above and below the cut. The James is a tidal river with (presumably) a very slight slope at low water. It is probable therefore that important advantages could be obtained by cutting off others of the great bends in this crooked stream. Those should, however, be first made which are nearest the mouth. The disastrous floods of the River Theiss, at Szegedin, in Hungary, have been due in part to the cut-offs made only in the upper portion of the stream, which precipitated

the flood waters upon the lower valley more rapidly than they could be carried off.

The comparison of the future of Salisbury on the Wicomico with the past of Glasgow on the Clyde, recalls some facts which may be of interest here. We may, however, be permitted to doubt if any depth of water in the Wicomico will ever make of Salisbury a second Glasgow.

Glasgow existed a thousand years before America was discovered. Its *second* cathedral was commenced before A.D. 1200. Its University was founded in 1450. In 1700 it was trading largely with America. In 1770 its importations were more than half those of the United Kingdom.

In 1740 the town council appropriated $500 to deepen the channel of the Clyde. In 1755 Smeaton was consulted. In 1768 Golborne suggested spur dikes and dredging. In 1799 Rennie advised shortening some of the spurs and connecting their ends by low training walls. "This was done, and much land reclaimed." In 1755 the rise and fall of the tide in neaps (at Glasgow) was barely perceptible. In 1836 the rise of neap-tides was 4 feet, and of spring tides 7 to 8 feet. In 1878 the range was 10 feet. When the improvement was commenced the Clyde at Glasgow was but 100 feet in width. It is now over 400 feet. The work has been performed by tidal scour, largely aided by the dredge, especially in more recent years.

Although the population of Glasgow has increased ten-fold in the last hundred years, it was already a large and most important commercial centre, with a trade which demanded the improvement of its line of access to the sea.

It is to be regretted that the author was not able to give his own experience with the work on Cape Fear River. The problem was a most difficult one, involving points of very great interest. The omission has been in great part supplied by the excellent paper of Captain Bixby.

The remark is approvingly quoted that we must "aid nature." True; but nature does not always work in harmony with our desires, and it is often difficult to determine what her efforts mean. We must not rashly assume them, but after careful study of *all* the circumstances of each case we must provide the conditions under which she will do her work, whether of creation or of maintenance.

It has been said during this discussion that the best works are not those which are constructed in the most solid and permanent manner, but those which give the best and largest results for the money expended. No principle has been more fully recognized than this in the whole range of Civil Engineering in this country. It was the necessity of covering a Continent with works of art with the limited resources of private capital in a new country that has developed the peculiarities of American Engineering. It has led us to a greater originality, to look to the end and the most direct means of attaining it, without regard to methods previously employed and under different conditions in the older hemisphere.

Mr. Burgwyn.—In discussing the paper before the Society, and the criticisms, or rather the enlargements, upon it, it appears as if sufficient stress had not been laid upon the essence of the paper, which is the lesson that an improvement can only be perfected after a prolonged experiment and study upon each individual subject. It is related of the late Professor Agassiz that his first lesson to his class in ichthyology consisted in giving them a fish to look at and examine for a week, and return with a written report. If the learned professor considered an entire week as necessary for the study of a single fish, it certainly ought to take many weeks, even years, for the study of so complicated a phenomenon as hydraulic action. The following table illustrates this :

Date.	Area in Square Feet.		Feet.	Ratio of Feet	
	High Tide	Low Tide.	Low Tide, Width.	Low Tide, Width.	Low Tide, Max. Depth.
1874	10 244	6 008	1 200	4.97	9.2
1877	7 899	5 841	588	9.93	12.1
March, 1878	7 905	5 840	590	9.90	11.9
September, 1878	7 294	5 544	500	11.09	13.1
December, 1878	7 123	5 443	480	11.34	13.2
November, 1880	7 426	5 785	469	12.33	16.0
March, 1881	7 501	5 856	470	12.46	15.2
May, 1882	7 359	5 776	450	12.84	16.1
April, 1884	7 472	5 911	446	13.25	16.5
June, 1885	7 572	5 968	445	13.41	16.3
March, 1887	7 594	6 028	444.7	13.55	16.4
August, 1888	7 578	6 026	443.4	13.59	16.3

This table gives the cross section of the James River at the crest of Richmond Bar, and extends over an interval of fourteen years. By an examination of the third column any one would say that the area at low tide was a constant, no matter what the width of the river may be. This appears to be true, but it took fourteen years' observation and over 100 000 figures to prove it. Nor does it necessarily prove that the low tide area would be a constant at any other section; and as a matter of fact, it varies as the mouth of the river is approached—increasing, of course.

Much has been said and written on the question of experimenting upon a river. If the truth were known, it is probable that many a physician has physicked a patient first in one way, then in another. Is it not analogous for an engineer likewise to experiment?

Again, in this discussion the town of Salisbury, Md., has been instanced, comparisons having been drawn with Glasgow, Scotland. While Glasgow may always be a much greater port than Salisbury, the improvement of Salisbury harbor reacts beneficially upon the whole country. A physician tells us that if one part of the body is diseased, it necessarily affects the entire system more or less. So if Salisbury

has been improved its products will be delivered cheaper into other markets, and hence produce amelioration much further reaching than the shores of the Wicomico.

Carrying analogy further, it is seen that in algebraic solution of equations between several unknown quantities, it is necessary for solution that there should be as many equations as there are unknown quantities. So in river and harbor improvement, there must be as many physical data, and constants, as the peculiar condition demands. These can only be determined by long and careful study and experiment. The chemist has to make many a test before he can determine the element.

Mr. HUTTON.—Careful and thorough preliminary study has been already noted as necessary to successful river work. The method by experimentation, or by trial and error, has been too well illustrated by the late Captain Eads to need discussion here.*

* See Transactions Am. Soc. C. E., Vol. XV (April, 1886), pages 276, 306-328.

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PLANT AND MATERIAL OF THE PANAMA CANAL.

By WILLIAM PLUMB WILLIAMS, Jun. Am. Soc. C. E.

READ JULY 2D, 1888.

The engineering world has been presented from time to time during the last eight years with accounts of the financial conditions and probable completion of the Panama Canal. These accounts have been in a large measure circulated in America for a desired effect by friends of the Canal Company, while other reports prejudicial to the Company have arisen from employees returning to this country out of health, or from disappointed contractors.

While on the Isthmus in the spring of 1888 I was afforded an excellent opportunity of making a thorough investigation of the whole work, and I think it interesting enough to American engineers to describe the methods of work and the plant and machinery employed in accomplishing the largest task of modern times.

A number of accounts of this work have been given by persons who gained their ideas from the windows of a passing railway train, and by hurried visits here and there to the different sections contiguous to the line of the Panama Railroad. This is so unsatisfactory a manner to see all systems of work, that I determined to go over the entire route, commencing

at Colon, by steam launch up the canal 15 kilometers, which portion is dredged 15 feet in depth, and then continuing the journey on different days by foot and horseback until I reached La Boca, the Panama end of the line.

In five weeks I traveled over the axial line of canal from Aspinwall to Panama, making a slow and careful examination. In many cases I secured statements from engineers of the Panama Canal Company, engineers of contractors, contractors, foremen and intelligent workmen, and in order to base an accurate opinion, I have seen all plant individually. It might be well to state that an American engineer need not go to Panama to learn how to execute effective and economical work, and it was most gratifying to contrast the expensive, slow and cumbersome methods employed by the French contractors with the lighter, more rapid and economical plant of American manufacture, used by contractors in the United States.

All plant on the Panama Canal is owned by the Canal Company, being leased to the different contractors along the line for 8 per cent. a year on the value of the machinery. This system has been expensive to the Company, as contractors not owning the machines would work them daily up to their maximum capacity, spending little or no time in repairs of minor working parts. These machines, from this want of care, would soon break down, and would be side-tracked, the contractor making a requisition for a new machine to continue the work. I saw hundreds of expensive excavators, rock drills, cranes, locomotives and hoisting engines side-tracked and condemned, only lacking a casting here and there to place the machine in working condition, but through negligence of the workmen rendered useless.

The atmosphere of the whole work is indicative of extravagance, the contractor regarding the Canal Company as a bank of inexhaustible supply, to be drawn upon at all times, and in some cases the claims for demurrages, breakages and losses made by the contractor are unfair, but are paid even though resulting in large loss to the Canal Company. In short, the climate is deadly, the contractor taking his life in his own hands when he goes there, and it is a question of making the largest amount of money in the shortest possible time in order to leave the country and enjoy the profits in a more healthful climate.

During the primary stages of the work the Canal Company placed orders for plant with the different foundries throughout France and

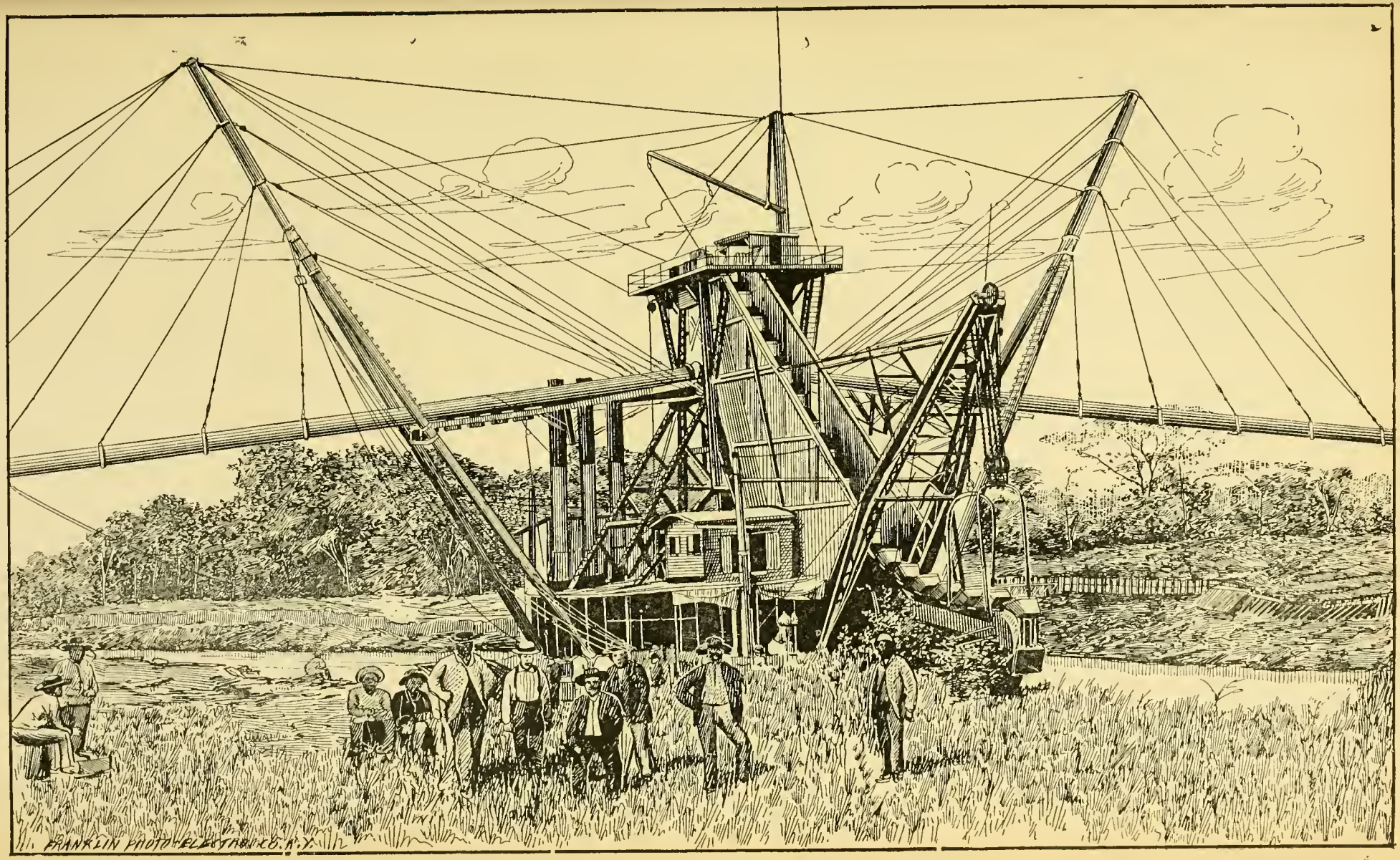


PLATE XLII. AMERICAN "HERCULES" DREDGE, CITY OF PARIS.

Belgium, which, being overtaxed, could not supply the machines fast enough. Consequently, the Canal Company made contracts with the several firms on the Isthmus to supply their own plant, to be bought by the Canal Company, paying the contractors 15 per cent. commission for their trouble.

This will account for the enormous amount of material on hand, such as hoisting engines, locomotives, cranes, dump cars, steel rails and dredges. This plant was purchased in quantities largely in excess of the amount necessary for constructing a sea-level canal, the contractors profiting by the excess of orders in their commissions; but now that the work is diminished by adopting a lock system, this large amount of plant will rapidly deteriorate by the action of the climate and be rendered useless. There are many varieties of the different kinds of machinery in use, but I have restricted my mention to those which have done the most satisfactory and economical work, in some cases describing plant which has not been intelligently used by the contractors.

The dredges in use are as follows:

American Hercules dredges, *French dredges*, *Belgian dredges* and *Scotch dredges*.

The American Hercules type (Plates XLII and XLIII) is operated by the American Contracting and Dredging Company, of New York, and their work all along the line on the Aspinwall side of the canal shows for itself as an evidence of what American contractors can accomplish in a tropical climate by their great activity and energy. I might also state that they are the only firm of contractors who have successfully carried out their work on time.

Their plant is as follows:

1. Count de Lesseps,	Low tower.....	All wood.
2. Nathan Appleton,	"	"
3. M. A. Slaven,	"	"
4. H. B. Slaven,	"	"
5. Dingler,	High tower.....	"
6. City of Paris,	High iron tower.....	Wooden hull.
7. City of New York,	" "	"

I find that this company has done good work by employing intelligent and skilled labor, mostly American, Irish and Swedish, housing and feeding their complement of men, and by attention to their welfare. The organization has been thorough, and the bonus system, by which the Captain and crew of the dredge are paid a certain amount of money *pro*

rata for having exceeded the average amount of work in any month, instills a high degree of ambition amongst the force. As to the amount of work done by the dredges I will mention the City of New York, at work at kilometer 14, deepening the channel of the canal from 15 feet. This machine was running nineteen to twenty-one buckets per minute, three-fourths full, three buckets to the cubic meter. The expansion of material in buckets was 30 per cent., occupying more volume in the buckets than in bank. I estimated in twenty-four hours' work it would accomplish 4 800 cubic meters. There are from forty-five to fifty-five men on these machines, distributed as follows:

1 Captain	$300 00	per month.
3 Assistant engineers	150 00	"
3 Firemen	70 00	"
3 Oilers	50 00	"
3 Diggers	65 00	"
3 Gypseymen	65 00	"
1 Steward and three cooks	75 00	"
6 Waiter boys	30 00	"
Seamen	50 00	"

These men are divided into three watches of eight hours each, the machines working night and day, only stopping to repair machinery. Sunday is usually occupied in replacing any worn-out material, replacing links, and anticipating any breakages in upper tumbler bars, boiler tubes, spud-gear, etc. From reliable records I find the average work to be relied upon is 100 000 cubic yards per month. The Dingler has excavated 237 000 cubic yards in one month, but it was in redredging material which had slid back into the canal, which was well lubricated and free from such obstructions as stumps and coral rock, allowing the machine to run at a higher speed, fearing nothing from disabling machinery or stoppages of flow of material through pipes from obstructions. I also find that the vibrations of the chain of buckets and links is reduced to a minimum when excavating in material not tenacious, allowing buckets to revolve twenty-five to thirty per minute.

The dredges of iron-tower construction have done satisfactory work, and are lighter in tonnage and of less draught than those of wooden structure, and much more stiff.

The first wooden dredges sent down to the Isthmus were not of the best quality. The Appleton was built in Philadelphia, and has done good work and been subjected to strains of rock and coral dredging, but her tower and hull are giving way more than is warranted had proper material been originally used. The tower being subject to the maximum

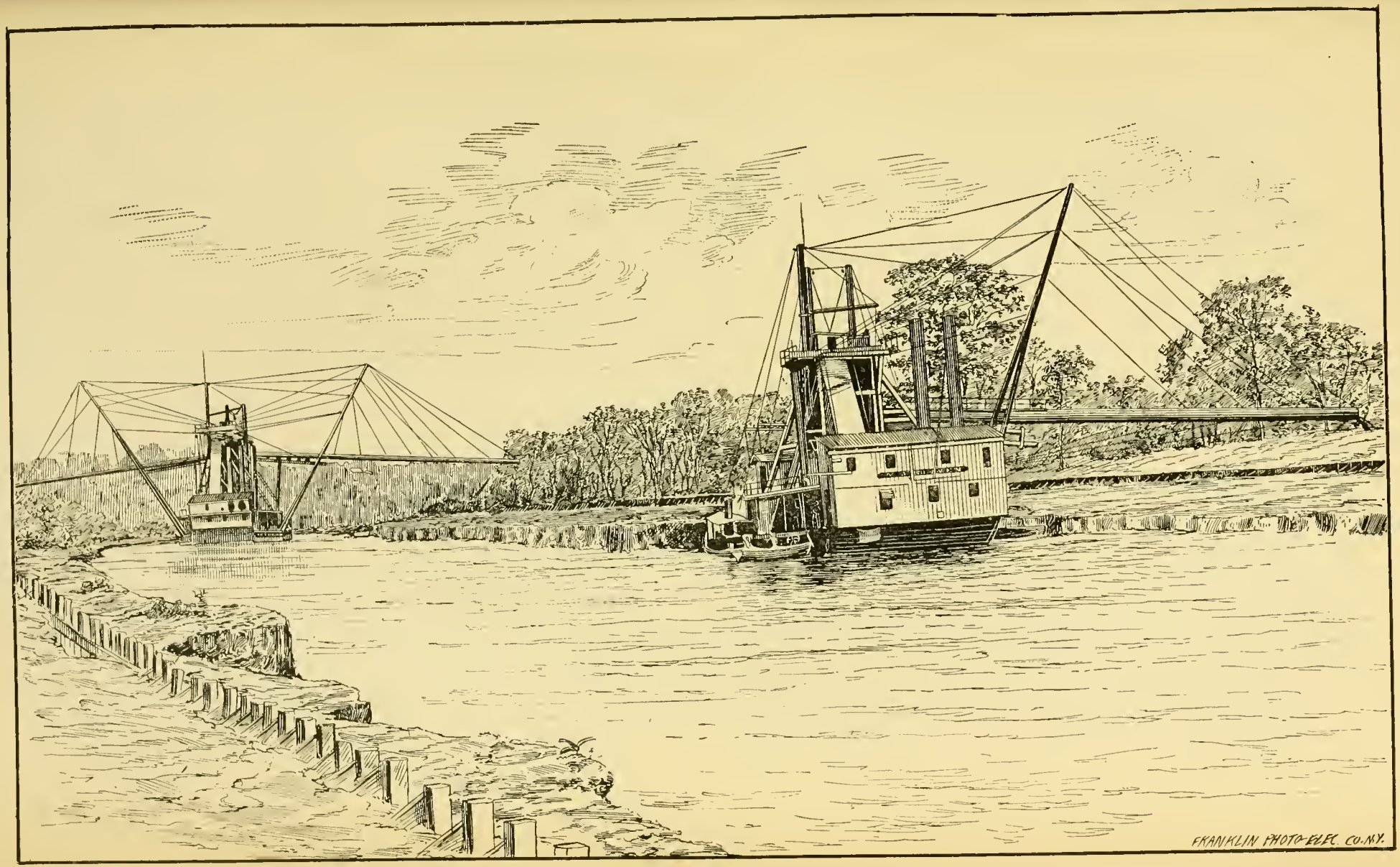


PLATE XLIII. AMERICAN "HERCULES" DREDGE, WORKING EIGHT METERS ABOVE SEA LEVEL.

strains, it was deemed advisable to build a machine with an iron tower, and in the City of New York and City of Paris I find a most admirable style of construction, the machinery running more smoothly and with less vibration than in the wooden tower. In the wooden towers packing pieces have been resorted to in order to build up solidly all spaces in the upper tower where shafting causes heavy strains and vibrations. Although perfect in work at first, yet from the elasticity of the wood and through friction the joints increased in size and the whole packing became loosened. In order to gain the immense strength necessary at so great height, 75 feet from the deck, the timbers are of large size, which increases the original cost and increases the tonnage proportionately. Sampson posts, bracing timbers, and packing pieces must be used as stiffeners. In the iron towers less total weight of material is used, reducing depth of flotation. The tower must be built to resist the strain of belt, producing a pulling strain from forward tending aft, and also the thrust of ladder, resolved into a horizontal thrust at tower and also tending aft. Thus these strains are all in the same direction and transmitted from strong tower to keelsons and distributed throughout the hull. The method of raising and lowering the ladder and the raising and lowering of the spuds may be improved by adopting hydraulic means. The spuds used as an axis upon which the dredge revolves are more effective than the control of movements of dredge by fore and aft guys as in Belgian, French and Scotch dredges, as rigidly holding the center of dredge and allowing revolution through a true arc, obviating the ridges which are caused by unequal work on the furrows in the other systems. The cost of the Hercules type has varied in the several machines from $100 000 to $120 000 complete at New York, tonnage on first machines being $5 000 to Colon and as low as $3 250 on the last machine. The same dredges could be built at the present time 15 per cent. cheaper and in a first-class manner. The Dredging Company have improved their machines from time to time, and even now, in the event of building a new dredge, could embody valuable improvements. I might say that the type of endless-chain-of-bucket dredges has been in use many years, and the Hercules type may be built at a royalty of $5 000 upon each machine, to be paid to the holders of the patent. I would especially note the importance of this type, as in using the high tower and long pipe of discharge, the capacity of work of the dredge is independent of the elements or human work other than on board of the machine. In other systems

emptying into clapets or discharging through pipes supported on pontoons, unavoidable delays arise from non-unity of action, delays in dumping clapets, and serious and unavoidable delays where a number of details of work must be successfully carried out to aid the performance of work of the parent machine. With the Hercules type this is illustrated, as night and day the machine is worked, the captain having a complete control of his machine and not being dependent on crews of men not under his supervision. This is also forcibly illustrated in visiting Belgian, Scotch and French dredges, machines capable of doing fair work, yet handicapped by minor details of superintendence and unsatisfactory work of their auxiliaries. The Hercules type consumes about ten tons of coal per day, and this item is a large one, considering the cost of transportation and trans-shipment of coal, and I would recommend, in future machines, that patent bridge walls and improved boiler construction be examined with reference to using less coal by more economical combustion of fuel. The buckets of the Hercules are a serious objection to their doing effective work in clay or tenacious material, as, on account of prismoidal form, they afford means of material adhering to the corners and bottom of bucket. I made visits to the Dingler, Appleton, M. A. Slaven, H. B. Slaven, City of New York and City of Paris, timing all work, noting quality of excavation in soft, sticky clay, hard clay, dredging material and sand, which I would estimate as follows:

	Cubic Yards per day.
Soft, sticky clay, buckets not fully emptying at upper tumbler	3 000 to 4 000
Hard clay	4 000
Sand	5 000

Allowing one day of each week for repairs of machinery, and all days regarded as twenty-four working hours. The estimate of cost of work must embrace the following items: Cost of dredge, towage and insurance to Colon; cost of erection and placing all machinery in working order; building and equipping a machine shop; supplying duplicate machinery at short notice in case of breakage; maintenance of crew; hospital service; cost of New York office and Colon office; erecting fence on banks to prevent sliding of material back into canal; leveling off tops of banks under discharge pipes when too high. In the first contract of the American Contracting and Dredging Company the price was thirty-five cents per cubic meter, and from estimates given me by

the company's engineer I may safely say that there was fifty per cent. profit in the work at that price. I have made investigation into all details of work done by this company, they having encountered difficult problems, dredging in low, marshy grounds where sliding of whole surface of marsh into the channel caused many delays, and the dredging of materials 7 to 8 meters above water level by erecting a system of dams. Their work, although performed satisfactorily, and to the great astonishment of the Frenchmen, is only a fair example of what our American contracting firms can accomplish in a tropical climate. Practically the whole work of the Hercules dredge is controlled by one man, who is stationed on the bow of the machine. A system of wheels at his hand connects with the different engines, namely, raising and lowering the lever; controlling the main engine, and velocity of revolution of buckets; the gypsey engine working the side guys; the spuds also being raised and lowered by tackles on hoisting drums. The digger may, at a glance, take in the situation, and use his governing wheels accordingly. Referring to the machine shops at Gatun of the Dredging Company, I would say they are so constructed as to do ordinary repairs of machinery at the shortest possible notice, and this plant cannot be dwelt upon too strongly, as in a region of no foundries and machine shops, a duplication of the principal parts of machinery should be attended to and complete shops built to supply these parts. The question of time is a most important element, and sending to the States for any supplies should be anticipated before breakage should occur. The report circulated in New York that the Hercules type of machine could dredge 16 000 cubic yards in one day of twenty-four hours arose as follows: at Gatun, in sandy and soft clay material, frequent sliding in the material occurred, so that the dredge was obliged to retreat to the rear and redredge the material. Material being soft and well lubricated, was easily attacked, allowing the buckets to revolve with a velocity of 25 or 30 per minute, without fear of breaking links by the vibratory movement occasioned by the upper tumbler, a square body going through an angle of 360 degrees. A word as to the preparation of land for dredges. The first contract for clearing and grubbing of right of way should be rigidly enforced, and all stumps, trees and obstructions removed from surface of ground, as these materials cause frequent delay in stopping up discharge pipes, although the buckets are powerful enough to disintegrate the mass.

Scotch Dredges (See Plate XLIV), are self-propelling, having steamed out from Scotland to Colon and also to Panama, passing around the Horn. Their boilers are of 200 horse-power, and their horizontal engines communicate power to a crank-shaft on which is a sprocket wheel. The upper tumbler shaft has also a sprocket wheel, and an endless chain communicates from the lower to the upper shaft, transmitting the motion. In heavy work these teeth break at frequent intervals. The ladder is in one section, requiring large construction of parts to gain the required strength for a long member. If in two sections it might be lighter and require less power to raise and lower. This dredge is more adapted for deep sea work than attacking new banks; it discharges into clapets and is controlled by fore-and-aft guys and side guys wound on friction drums. Its draught is 7 to 8 feet, and it burns 6 tons of coal per twelve working hours. I visited three Scotch dredges in the Bay of Panama, but their work in discharging into clapets averaged only 200 yards per hour on account of encountering coral rock. In ordinary work they accomplish 2 000 to 3 000 cubic meters per day of twelve hours.

The French Dredge (See Plate XLV and XLVI), is the principal dredge in use along the line of the Panama Canal. There are different sizes, the one most in use being 100 feet long by 30 feet broad, and having a draught of 7 feet of water. The hulls and entire machine are constructed of iron, in sections, in France, shipped to Colon and transshipped at different points along the line where they are to be used. The cost is approximately $115 000 at Colon, not including cost of erection, which has been an expensive work at Panama, some engineers estimating the cost of erection at 35 per cent. on original value. The tower is quite low, the elevation of hopper below upper tumbler being only 20 feet above water level. The ladder is in one section, supported upon axis in tower, and varies in length to the use of dredge in attacking new banks, or in deepening channels. The buckets are of iron, wrought in one piece, the links being an integral part of the bucket, which I do not regard as an economical construction, or any stronger than if in a separate piece, (the Slaven bucket has links bolted to the flanges on the back, so that in case the link becomes much worn it may be easily replaced by a new link without necessitating loss of bucket, which may be in perfect order). The power is derived from a vertical engine having three pistons, which act directly upward on a crank shaft, which has a gear-wheel

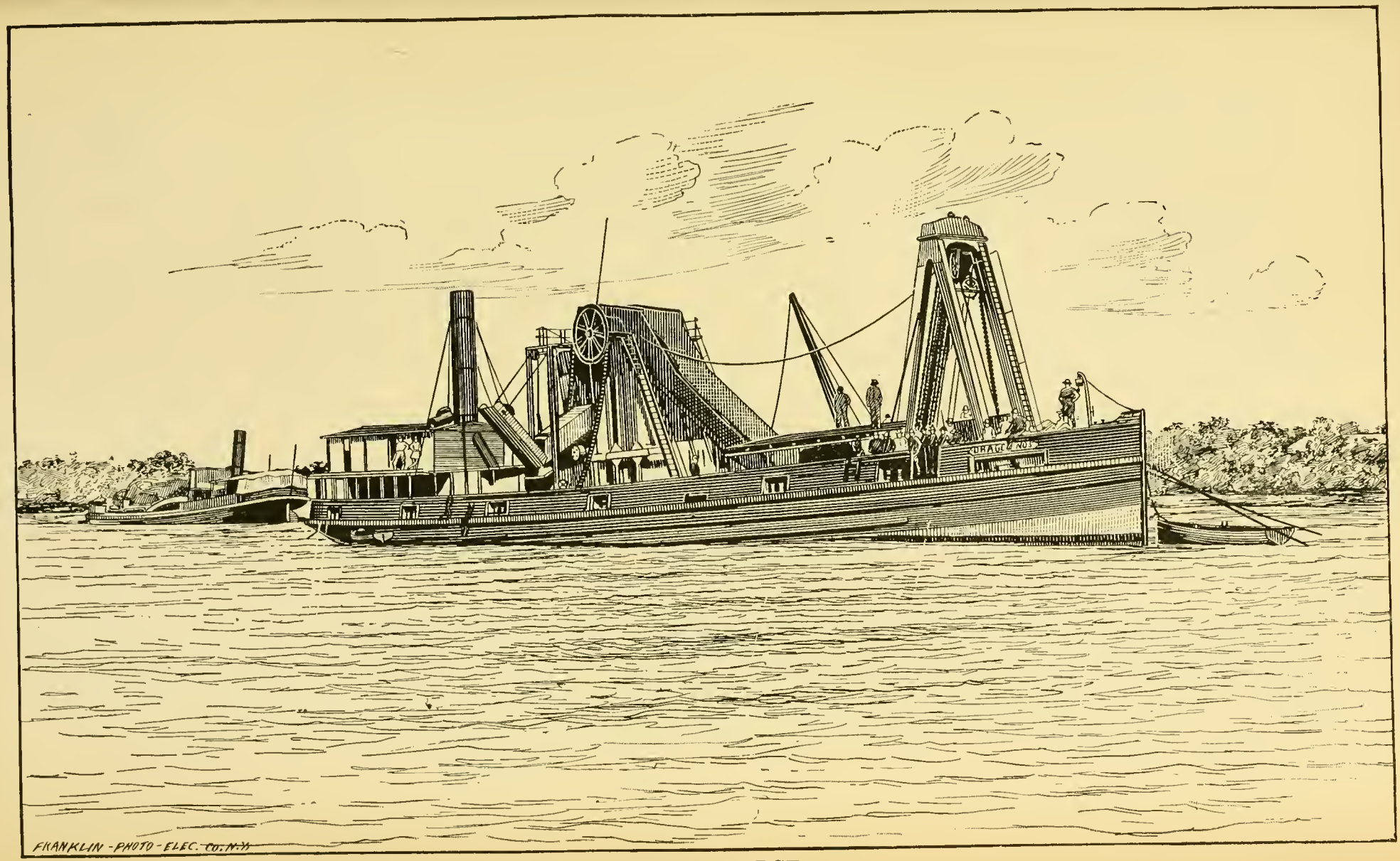


PLATE XLIV.

SCOTCH DREDGE.

at either end, and large balance wheels. These gear-wheels connect through two other gear-wheels to the upper tumbler shaft, thus giving a positive power, and when machine is dredging in rock, no slipping occurs, as in a belt connection. The engines are 180 horse-power in this sized dredge, and it forms a most powerful machine, so that in attacking hard-pan or loose rock it receives such a force as to accomplish its work when buckets and links do not break. In ordinary work in sand, gravel, clay and loose material a positive force is not necessary, as in rock work. The large belt from a horizontal engine connecting with a gear attachment fitted with a tightener pulley, increasing or diminishing the tension, has given good satisfaction, and controls the movements, except in rock work.

The dimensions of a French dredge of large type are as follows: Length, 120 feet; breadth, 28 feet; depth, 10 feet; draught, 7 feet; depth of working, 28 feet; sheer fore and aft, 10 inches; rise of deck, 6 inches; height of discharge above water-line, 20 feet; height of top tumbler above water-line, 26 feet 6 inches; width of bucket well, 5 feet 3 inches; frames, 4 inches by 3 inches by $\frac{3}{8}$ inch, 2 feet apart, with reverse angle irons 3 inches by 3 inches by $\frac{3}{8}$ inch, in alternate frames; plating of bottom and bilges, near well, $\frac{7}{16}$ inch; plating otherwise, $\frac{3}{8}$ inch; plating of sides, $\frac{3}{8}$ inch; plating of well, $\frac{3}{8}$ inch; deck beams, bulb iron, 8 inches by $\frac{3}{8}$ inch, with double angle irons $2\frac{1}{2}$ inches by $2\frac{1}{2}$ inches by $\frac{3}{8}$ inch; floors, 12 inches by 4 inches; angle irons, 3 by 3 by $\frac{3}{8}$ inches; length of bucket ladder between centers, 64 feet 6 inches; capacity of buckets, 16 cubic feet; diameter of pins, $2\frac{7}{8}$ inches; high pressure cylinders, 17 inches diameter by 24-inch stroke; low pressure cylinders, 34 inches diameter by 24-inch stroke; air pump, 10 inches diameter by 15-inch stroke; circulating pump, 10 inches diameter by 15-inch stroke; boiler diameter, 10 feet 6 inches; boiler length, 9 feet 6 inches; boiler heating surface, 900 square feet; boiler working pressure, 80 pounds per square inch; cost at Colon, $115 000.

One of these machines of large type has done valuable work at the Mindi Cut, near Gatun, on the Panama Canal, in broken rock, stiff clay and hard-pan. The material excavated in buckets is carried up into a hopper, discharged with water, pumped up hydraulically sufficiently to discharge it into self-dumping steam clapets alongside. The capacity of these dredges is variable in the extreme, no one machine having done a large amount of satisfactory work. A fair estimate, where no demurrage

occurs from want of sufficient clapets alongside to allow the dredge to steadily discharge through twelve working hours, is 200 to 250 yards per hour. At La Boca, in Panama Bay, a dredge of this type was running twenty buckets per minute—three buckets per cubic meter when full; these were half filled. Coal consumed was about 10 tons per day, and working a crew of twenty-six men, composed of sixteen sailors at $50 per month and ten men in the Engineering Department, the cost of maintenance being $188 per day, excluding breakages. One most important objection to these machines is the number of skilled men necessary to work the machine satisfactorily. The captain or digger stands on the bow of the dredge and takes in the situation, communicating by word his orders to the individual engineers, one in charge of main engine, one in charge of gypsey engine, controlling the side guys; another controls by engine the fore-and-aft guys, while another engineer controls the ladder, raising or lowering it when necessary. In one case in question these positions were filled by Jamaicans, Columbians, Greeks and Italians, no two men being of the same degree of intelligence, causing unnecessary delay from loss of time and non-unity of action; while by a system of levers on the bow, connecting with each engine, the digger would be able at a glance to control his machine as completely as an engineer of a locomotive.

I regard these dredges, with this improvement, able to do effective work, and with a high tower and long discharge pipes equal to the Hercules, as their engines are well built, and with good use of power well applied, their ladder and buckets built stiffly, and strains well distributed through tower and hull. The system in all foreign styles of dredges of constructing the ladder in one section is more adapted to deep-water dredging than to attacking new banks, while the double ladder of the Hercules type will excavate higher banks with less power applied to raising or lowering the ladder. By the control of the dredge by fore-and-aft guys and by side guys, the work is not as evenly executed as when the dredge revolves upon a central axis or spud, as in a machine digging through an arc of 45 degrees it necessitates the slacking off of both forward and after guys and of one side guy, and "taking up" the other side guy, and as this work is done by several men, and with no absolute precision of action, it is seldom that a dredge is moving to and fro in concentric furrows.

French Dredges are also in use where the power is applied through

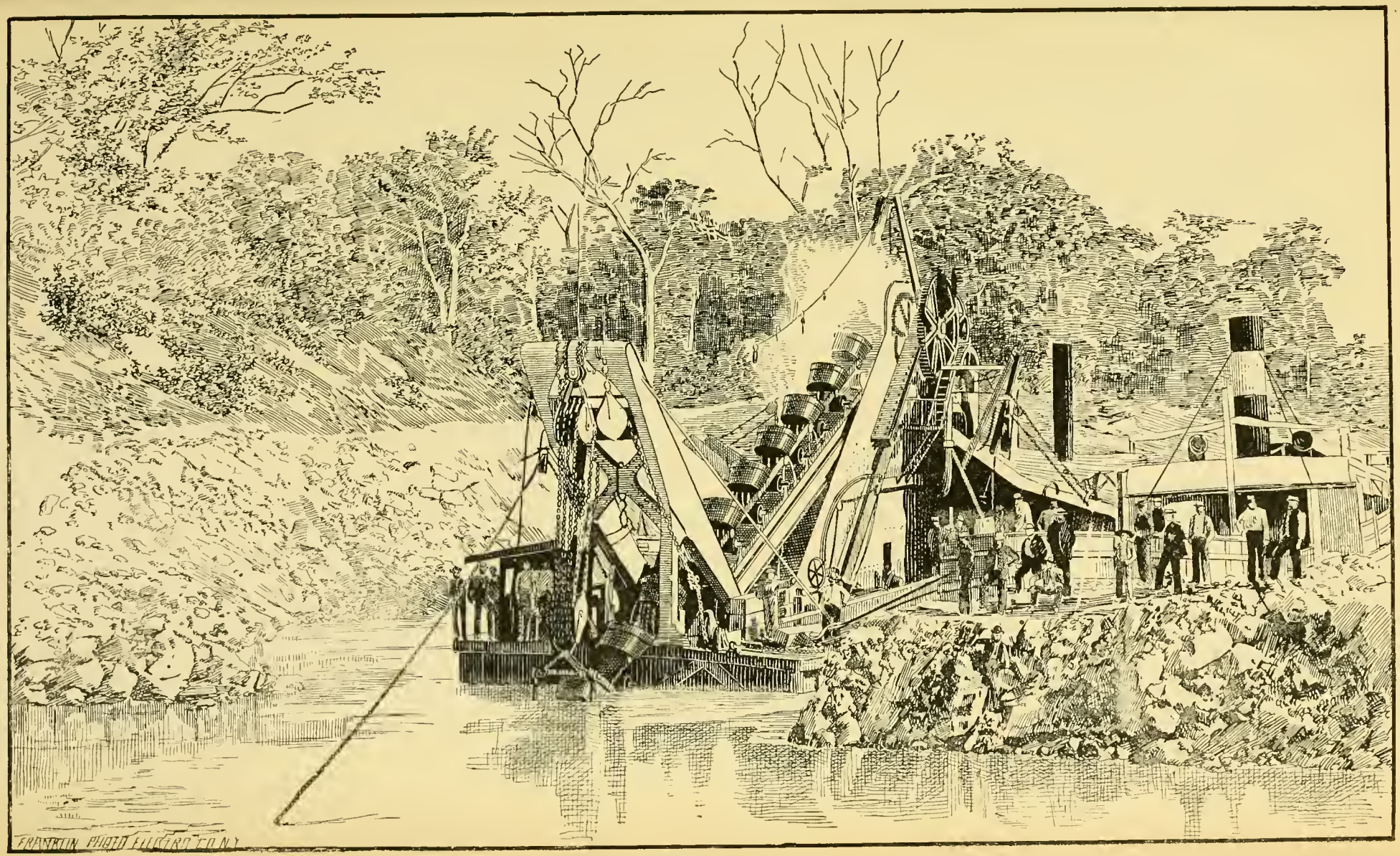


PLATE XLV.

FRENCH DREDGE.

an endless chain fitting over sprocket wheels on the crank shaft and upper tumbler shaft. I find by this system that these teeth break frequently, and it would seem to be most advantageous to use in dredges designed for heavy work a vertical engine connecting directly to gear wheel gearing to upper tumbler shaft. A marked contrast is shown in all machines of foreign construction in being so much heavier in all details than American machines, and in the ladder two sections might have been used, each one having been built lighter and much more stiff than if building it in one section, thus necessitating a ponderous frame to resist buckling of the long length. I submit a record of work of the French dredge at La Boca, April 16th.

Character of material, obdurate clay 8 to 10 feet deep.

Clapet No.	5.	Capacity	233	cubic meters,	filled by dredge	1 hour.
"	8.	"	"	"	"	45 minutes.
"	5.	"	"	"	"	55 "
"	8.	"	"	"	"	40 "

Deducting sixty for amount of water in material and expansion, total 173 cubic meters; and I might add ten to fifteen minutes were lost each time a clapet came along side, made fast, and dredge commenced working. At Tavernilla, on April 17th, I examined a French dredge at work, attacking a bank 30 feet above water level. This dredge was of 200 horse-power, and by undermining the bank, causing it to fall into the bed of the stream, it was able to accomplish its work. The method of discharge was hydraulic, as follows: in the neck of the hopper two gratings were securely placed at right angles, making squares 2 inches each way. The material discharged from bucket and falling onto grating was broken up a certain amount by two 8-inch streams of water, pumped hydraulically and directed in their discharge into the hopper to strike the falling material, which became disintegrated when going through the grating. The material then flowed outboard into a closed box at the lower end of the hopper. A scow made fast alongside of the dredge was supplied with boiler and horizontal engine, communicating motion to a centrifugal pump with a 14-inch pipe; a vacuum being formed, the disintegrated and lubricated material in the box at bottom of hopper was taken by suction through the pump, and forced through the corrugated iron discharge pipes, 14 inches in diameter, and supported on pontoons moored in mid-stream; then forced in this case over onto the shore and

discharged 30 feet higher than the pump, which was 15 feet above water level, so the elevation of the discharge was 45 feet above water level, and 600 feet distant. They claim to be doing 1 200 cubic meters per day, which was possible when not detained by obstructions. The objection to this system is, that by contact of material with blades of pump oftentime bolts, snags, stones and obstructions of all kinds large enough to go through grating lengthwise would break the blades of the pump. Again, the capacity of dredge running twenty buckets per minute, and discharging material into pipe, in order to work effectively, would require discharge pipe of 20 inches in diameter, instead of 14 inches, as usually 60 to 80 per cent. of diameter of full discharge pipe is water, and the remainder material. The system of pontoons employed for support of pipe was successful. Four cylindrical air-tight boilers were constructed of iron, each 5 feet long by 3 feet in diameter. They were floated horizontally in the water and connected rigidly side by side, by wooden braces, and the raft thus formed supported a vertical X brace at each end of the pontoon. The discharge pipe was supported in the Y of the brace. Between the pontoons was a flexible joint 3 feet long, formed as follows: A sleeve of leather was made, the same diameter as pipe; this leather was covered with strong copper wire cloth to make the joint more rigid, and the whole riveted at each end to flange of corrugated iron, of same size as discharging pipe, and this point was made fast as an intermediate, allowing flexibility. This form is not as expensive as ball and socket joint connection, and with leather of sufficient thickness will stand high pressure 100 to 120 pounds to the square inch.

Where action of dredges is dependent upon discharge through pipes supported upon pontoons, circumstances arise which will seriously impair a regular quantity of work. Pontoons must be controlled with guys and end of discharge pipe moved from place to place when bank arises from accumulation of discharge material. The discharge pipe must be lengthened or shortened, and transported forward or aft, as the dredge moves forward in her channel of work.

Hydraulic Dredging.—The method of hydraulic discharge in use at Tavernilla I have described, and at the same place I saw in process of construction an auxiliary to the French dredge. An iron barge 60 feet long by 35 feet broad contained in the stern three horizontal engines of large type, stroke 3 feet, diameter of piston 20 inches. The piston was

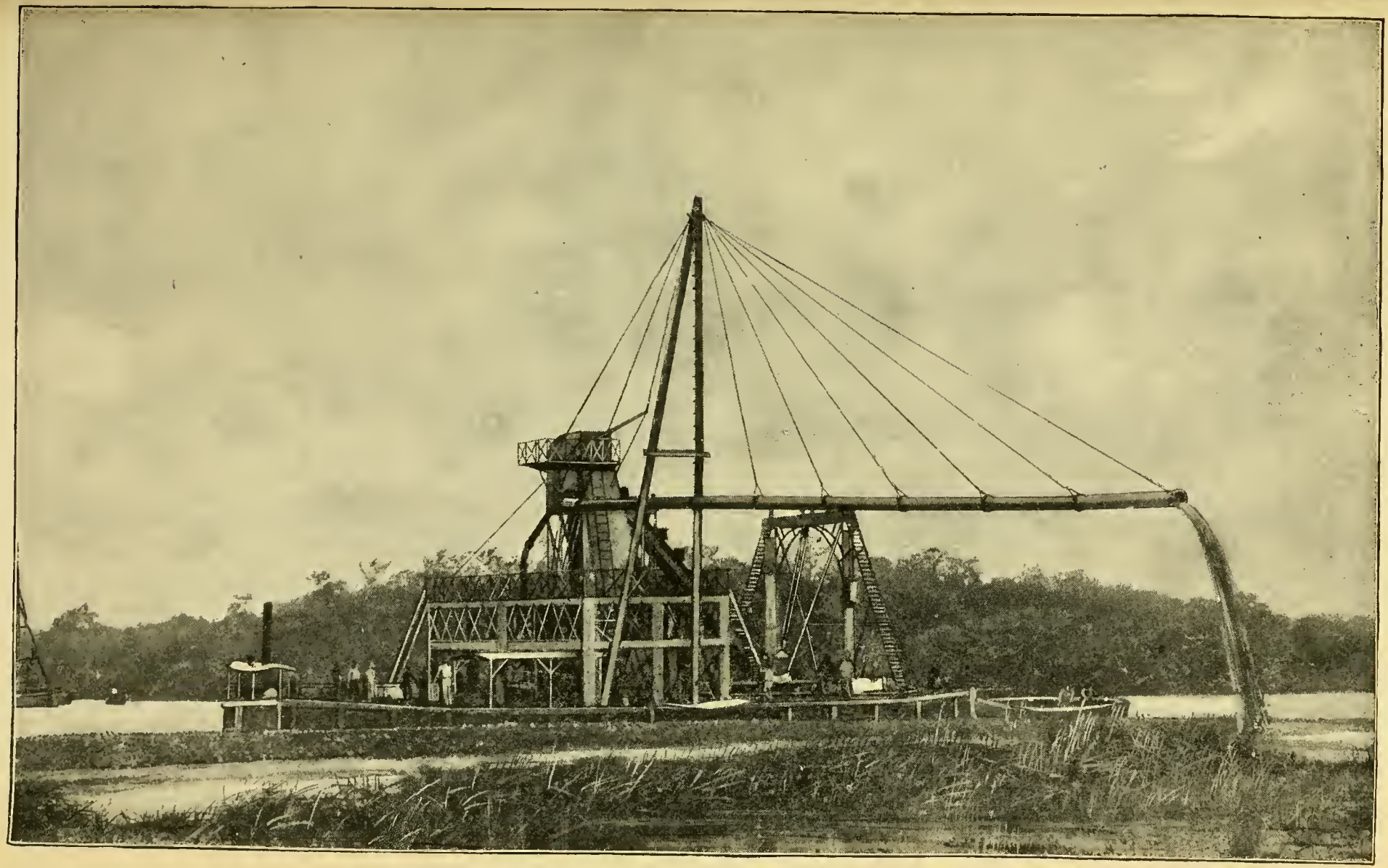


PLATE XLVI. FRENCH DREDGE WITH LONG DISCHARGE PIPE.

elongated and connected direct with cylinder of water pump. The three water pumps received their supply of water from the side of hull, and under high pressure from the cylinder it was forced forward to the hoppers. Amidships in the barge were built two hoppers with large cylinders and a small bottle-necked opening, receiving the discharge of material from the hopper of dredge, the material being well lubricated by water thrown into hopper of dredge through two 9-inch pipes. At the neck of hopper are two iron gratings, placed at right angles over each other, disintegrating the material which falls into the hopper. The outlet is at the bottom of hopper of 20 inches diameter and the entrance pipes for water force are three in number, entering the hopper two at the bottom, and on the quarter opposite the outlet pipe extending the stream to strike the material and drive it toward the outlet ; also one pipe entering from above opposite the outlet and inclined in a downward direction ; these three streams produced would meet at a radial point, the objective place of outlet of material. This gives the material a great force, depending upon the amount of pressure of the engine, and as the "chute" from the hopper of tower to hopper of barge is a closed pipe and filled with a continually falling mass of earth and water, it practically fills the full diameter of the pipe, making it air tight, so that with any induced pressure at the hopper of the barge, the only outlet of material would be through the bottom of the hopper, then out on shore through pipes located on pontoons. The advantages of this system are as follows : material does not come into contact with parts of pump, and in extreme cases when gravel work is to be excavated, the grate bars are removed, thus allowing all material to pass through the hopper when the size of obstruction is not larger than the cross diameter of discharge pipe. In dredging high banks, where the discharge must be made at some distance vertically and horizontally from the dredge, this is the best method in use. Also for harbor work when not affected by swells of ocean or surf, the endless-chain-of-bucket type attacking the material and discharging into hopper and from there floated to the shore by hydraulic discharge may be most expeditiously used. I would recommend that as this plant is quite expensive, the auxiliary barge, costing $20 000, should be constructed separately from the hull of dredge, and may be used with hydraulic or Hercules dredges by proper connections.

In sand dredging the buckets revolved at a speed to fill 15 per

cent. of diameter of pipe, the remaining 85 per cent. being filled with water, which must be at higher pressure than for clay work, as each particle of sand sets up for itself, and unless kept moving at a uniform rate will form sand bars in the pipe.

Clay should be disintegrated before going through the discharge pipe, as it forms balls uniting with sand, gravel or obstacles, and by rolling through pipe, hardens, and I have seen lumps issuing from the pipe of nearly three-quarters the diameter, thus blocking up the steady flow of water and material.

Belgian Dredge. (See Plate XLVII.)—This type of dredge used on the Panama Canal is quite similar to the French dredge, deriving its power in the same manner by sprocket and chain connection. The horse-power is two hundred, fitted with three horizontal fire-return tubular boilers, and two horizontal engines, whose pistons connect with crank shaft, on which is a wheel. I find in this system of deriving power that the teeth are constantly breaking. The revolution of the upper tumbler is not with constant velocity, as there are four distinct movements or pulls each time the square upper tumbler goes through the arc of a circle. When the movement is constant then a toothed wheel and chain connection might be satisfactory, but I do not regard it as such in dredging tenacious material when a positive force more directly applied is necessary. The Belgian dredge discharges on either side into clapets, and the velocity of buckets was from fifteen to twenty per minute, each holding one-third of a cubic meter. I did not see any improvements on the machine other than were embraced on the French dredge. The cost was less than the French dredge on account of less tonnage, and I was informed they could be landed on the Isthmus at $90 000 each; the percentage of construction varying largely, whether they were "set up" at Colon or some miles distant on the line, requiring shops and quarters for men to be erected at an additional expense. The complement of men ran from thirty to forty, and in French, Belgian and Scotch machines were arranged in only one watch, and these dredges only worked during the day, on account of liability of clapets running aground in steaming to and fro from the dumping ground at night from unsafe pilotage.

Rock Drills.—A large consignment of the Ingersoll type of rock drills was sent down to the "Société de Travaux Publiques" for use at Bas Obispo and at Emperador. These drills were in use only a few

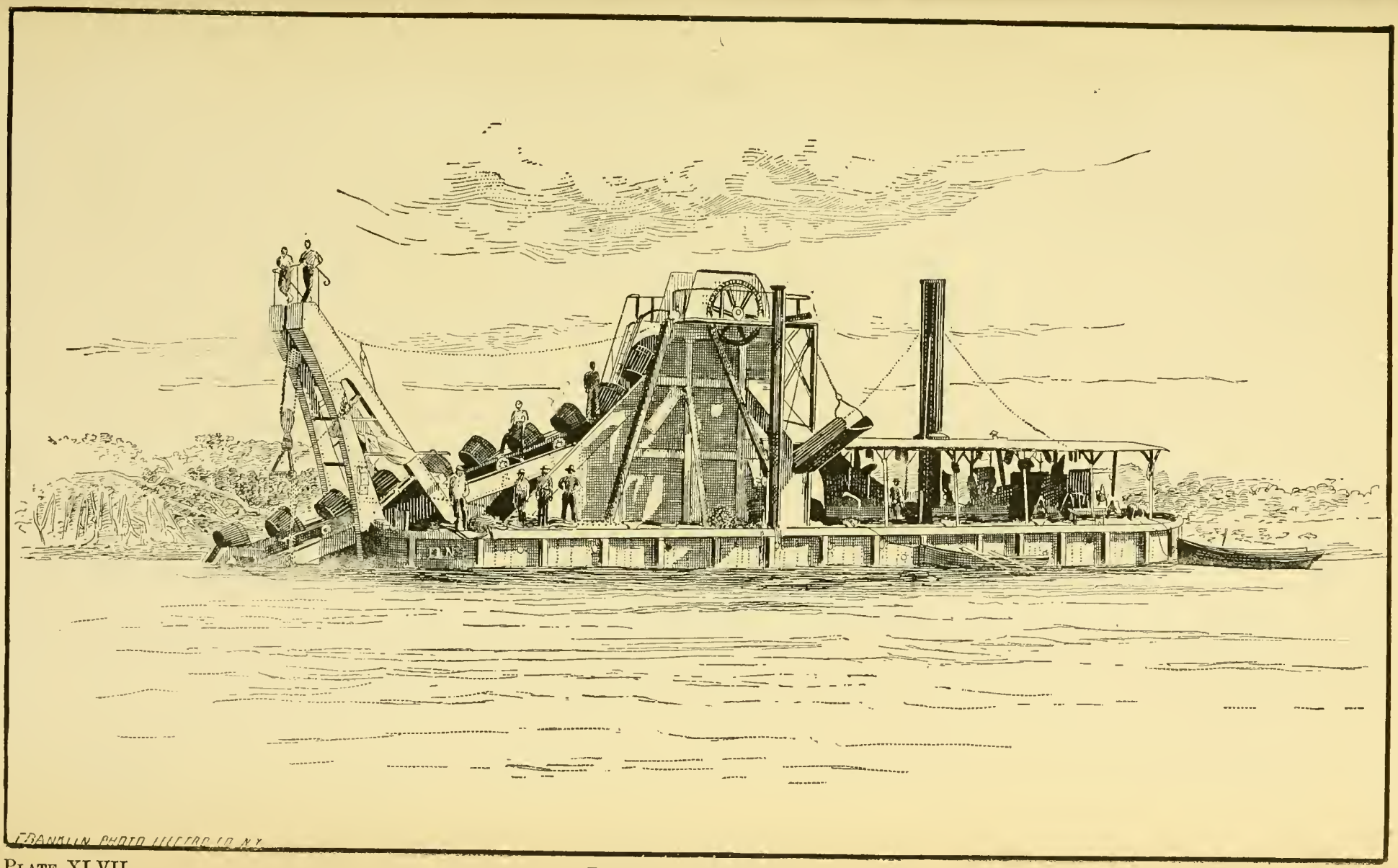


PLATE XLVII. BELGIAN DREDGE.

frame of the drill and is driven down into the bottom of channel, and the drills are then lowered and commence work inside of this water-tight sleeve. The four drills make their holes, when the drill is withdrawn, and the charge put in and wired, the wires brought up out of the water and made fast to the deck. Then the frame is moved forward on its tracks a few feet and four more holes are drilled and loaded. The hull is controlled by fore-and-aft guys and side guys, thus limiting its work over a certain territory. When the area over which the open slot floats is charged, the hull is moved, and successive holes are drilled, when the boat withdraws a safe distance and the blast is made. The coral rock at Panama and Colon has been disintegrated in this manner and when broken in small pieces has been attacked by the dredges. I regard this means as quite successful in deep submarine work, and economical. The cost of an iron hull would be three to five thousand dollars with boiler and fittings of drills, and entire plant would not be over $2 000 more.

Blasting.—Besides the holes of small diameter used in excavation of rock, larger shafts are resorted to, in cases where the material is of volcanic formation and porous, causing a charge to blow out in the cracks of the different strata. A hole 1 meter square is excavated some 10 meters in depth; at the bottom of this shaft two chambers are extended in either direction, 1 meter square and 2 meters each, long. These chambers and the shaft are filled with barrels of powder and dynamite. At a distance of 5 meters below the surface of the ground cement is used as a filling, and after setting forms a solid mass of artificial stone to the surface of the ground. A number of these shafts are made and all fired at once. This immense explosive force shatters the surrounding mass, which may be easily attacked by the workmen with picks, loaded into buckets, and swung around and dumped into cars by cranes. In soft rock this has been the only successful means of working, but is costly on account of the large quantity of powder used in filling and loading the shafts.

The large works in hand are all accomplished at the present time by the use of churn drills, the ignorant workmen using drills from 6 to 15 feet long, of different diameters. They receive twenty-five to thirty cents per linear meter for holes of 6 to 8 feet in depth or under, and where the holes are over 8 feet in depth they receive from thirty to forty-five cents per linear meter, the quality of rock regulating the

schedule of prices largely. These prices include the labor of loading and firing the holes, the cost of material to be furnished by the contractor. The cost of transportation in using rock drills, on account of weight of boilers and drills and frame, becomes an objectionable feature, and the general policy has been, in the absence of intelligent men, to pursue such simple means and appliances that when the ignorant force is left alone and without oversight they can accomplish satisfactory work, hence the substitution of the churn drill for our effective mechanical system.

Steel Rails. (See Rail Section, Plate XLVIII).—For construction purposes a rail weighing from 40 to 55 pounds per yard, and with base equal in width to the height, would be of sufficient strength, where the maximum weight at any one time is a French excavator of 36 tons, whose base is 30 feet; ties 2 feet from center to center. I find that one most costly mistake has been made on the Panama Canal by ordering for construction purposes over 120 000 tons of 70-pound steel rails, of such dimensions that the height largely exceeds the base. The Belgian locomotive weighs 30 tons, having three drivers and a rigid base of 13 feet, so that in curves of small radius or over 8 degrees the overturning movement is so great as to cause rails to spread, drawing the spikes. This has caused many derailments, and especially on construction tracks which are improperly ballasted the rail and tie must stand the whole strain. Then again on dumps or terraces where tracks must be frequently moved from side to side, it requires a larger force of men to do this work than if handling a 50-pound rail.

Steam Engines.—The following firms are largely represented by their engines upon the Isthmus: Société Cockerill; Société Belgique; Société Anonyme de Couillet Belgique; Rogers Locomotive Works; Baldwin Type of Burnham, Parry & Williams. The Belgian locomotives are quite similar in construction, weighing 30 to 40 tons, supported by three drivers on each side, connected rigidly, the middle driver having no flange. Their capacity is hauling twenty-four loaded dump cars, weighing 10 tons each, up a three per cent. grade. They consume about three tons per day, and require a crew of three men and engineer. Wages of engineer, $4 to $5 per day; wages of men, $1.50 to $2 per day.

The foreign machinery is much heavier than the American, burns more coal, and gets much more easily out of order. The foremen and

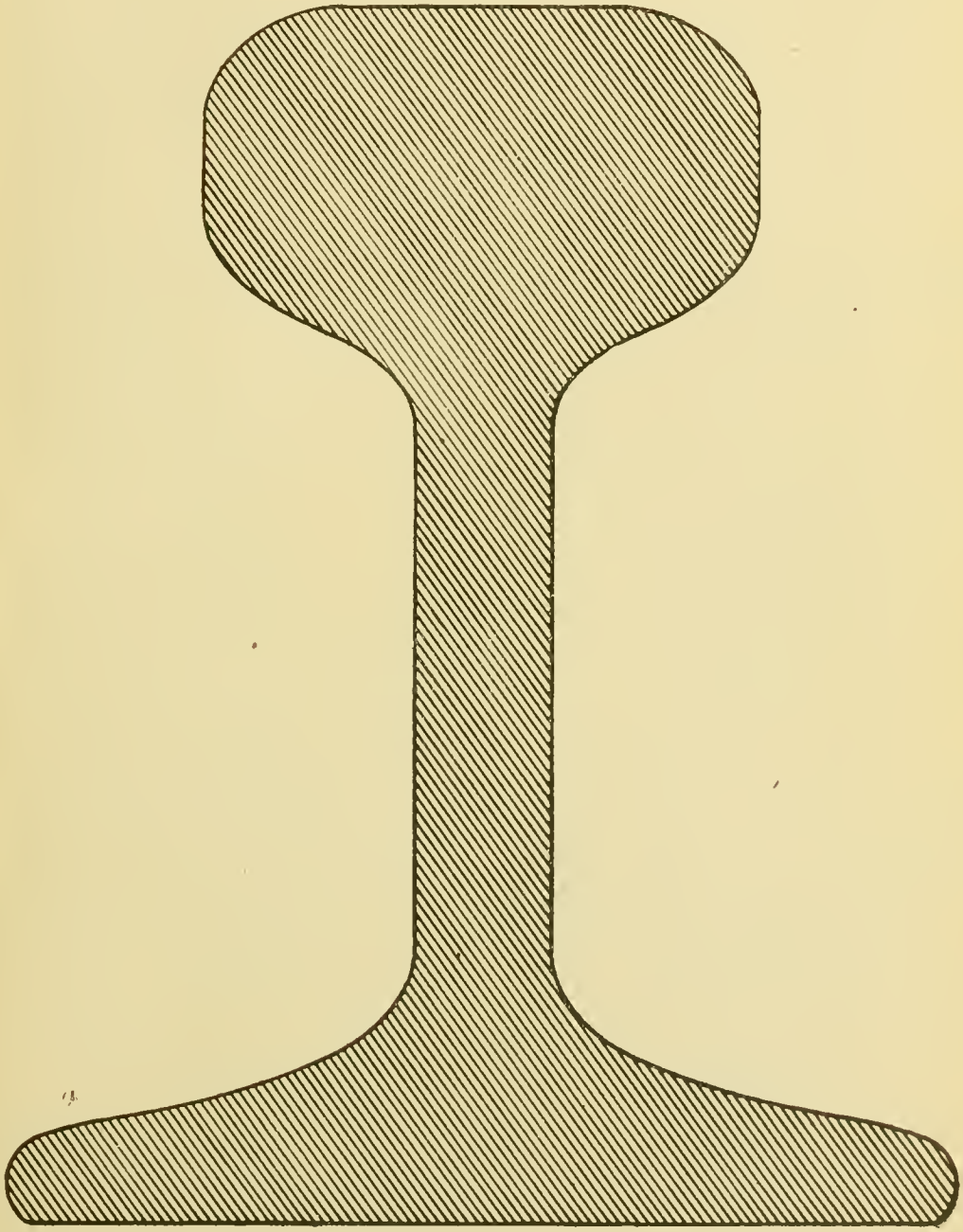
engineers on the line of canal prefer the Baldwin locomotive, manufactured by Burnham, Parry & Williams, for rapid work and easy control of parts. The Rogers engine is also in good form, and both American engines consume less coal than those of foreign manufacture. The cost has varied from $12 000 to $15 000, delivered at Colon.

De Cauville Cars (See Plate XLIX).—The De Cauville system of pan cars includes two styles, namely, the De Cauville and Baillard plant. The Baillard cars are of box shape, holding ½ cubic meter. The iron frame of truck is solid and holds the boxes of the wheels. The box is of iron, having a semi-circular arc at each end, resting upon a grooved track, and in dumping the body of the car travels on the arc, throwing the box through an angle of 45 degrees. Their weight is 500 pounds, and cost $50 to $60 apiece. The gauge is 20 inches.

The De Cauville car is quite similar in the construction of truck, but the body of the car is shaped like a triangle, base uppermost, and supported by two lugs at each end, fitting into an upright frame made fast to truck. The center of gravity is in equilibrium when the car is full and it may be easily dumped by throwing the body through an angle of 45 degrees.

This size runs two and a half to three per cubic meter. The skeleton track is of 20-inch gauge and constructed in sections of 15 feet, the iron ties being made fast to the rails and easily transported in small space in large quantities. This plant is not in use with mules or horses on the Panama line, and I have seen numerous instances where they were used by the natives in executing task work, when they would receive thirty cents per cubic meter for filling and pushing cart to dump, several hundred feet, when by mule or horse labor and train of six or eight cars to each animal they could be hauled to place more economically. This plant is far superior to any invention in the States as a light and economical means of attacking preliminary work. In preparing cuts for excavators and in surface work where irregular territory is to be smoothed down for location of heavy construction tracks for steam excavators this plant is expeditious.

Broken Stone.—I found a French stone-breaker at work at Pedro Miguel and at Buhio of larger and more massive construction than a similar American machine. The broken stone for the locks at Buhio, San Pablo, Bas Obispo, Emperador and Paraiso is all task work and executed by hand, the men receiving $2 per cubic meter, and in many in-



SECTION OF RAIL USED FOR CONSTRUCTION ON

THE PANAMA CANAL.

PLATE XLVIII.

stances quantities of the stone broken during the last year has been of such quality that when exposed to the atmosphere it became disintegrated and crumbled. In nearly all cases of rock excavation for the locks at San Pablo, Buhio, Bas Obispo, Culebra and Paraiso solid stone is not met in the upper strata, and at the quarry of the Panama Railroad, at Buhio, is the only large quantity of stone suitable for masonry and concrete.

At several points along the line I noticed the broken stone had been whitewashed, and inquiring the reason, was told that the Jamaican laborers, during the night, would steal broken stone from one another's piles, which has been paid for at $2 per cubic meter by the contractor, the laborer turning in the stone to be paid for over again, and the contractor was obliged to adopt a whitewashing system to distinguish work paid for from that not accepted.

Submarine Rock Work.—In connection with the submarine rock drills may be mentioned the grapplers in use on the Panama line. They are constructed very much on the same principle as cranes, and the hull is large enough to support a 10 horse-power boiler, horizontal engines and crane swinging through an arc of 270 degrees. Attached to the crane are the grappling irons, operating like a clam-shell dredge, and these teeth are lowered to the required depth, grappling the broken rock, which is hoisted to the surface, the crane swinging the load and dumping into a barge alongside. The hulls of these machines are constructed of iron, in sections, the hull being 40 x 18 x 6 feet deep, and have been largely used in removing coral rock on the Colon side, which has been first broken up by drilling. These machines, on account of size of hull and easy means of handling, are able to move in shallow water and work in closer quarters than if cranes and grapplers were applied to each dredge, as in the Hercules work, and in using them separately from the dredge much time is gained, as the dredge can keep on with her work, and independently of rock which may be afterwards excavated.

Cranes. (See Plates L, LI, LII.)—Rock work in nearly all cases was executed by the use of steam hoisting cranes, which were used in large variety and of the following manufacture :

1st. J. Voreez Ainé, Nantes, 1887 ; 2d. Stothert & Pitt, Bath, England ; 3d. Applegate Bros., London, England.

The first named was of the "goose-neck" style of crane, which was supported on four wheels, car of 5-feet gauge, the whole crane weighing

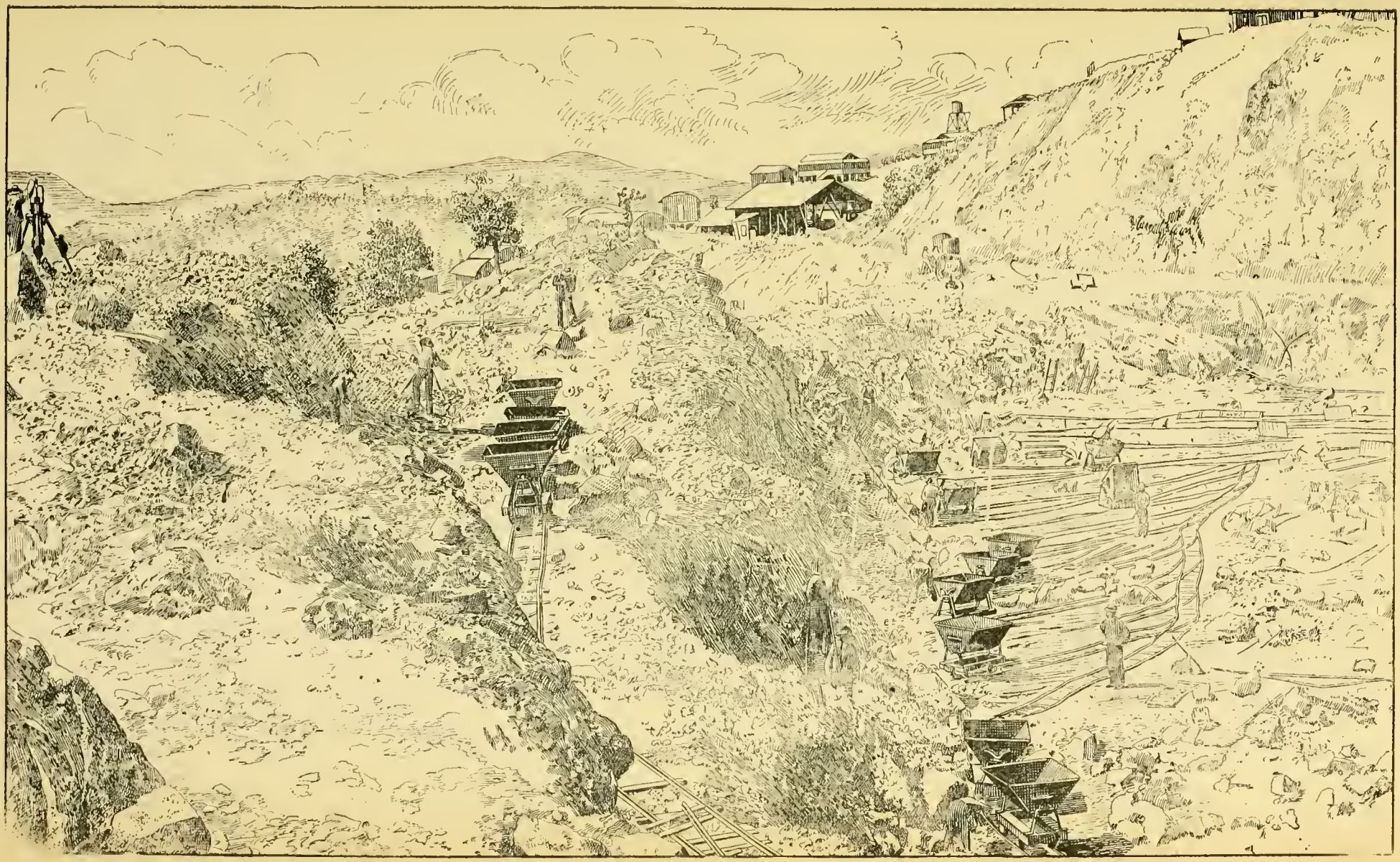


PLATE XLIX. DE CAUVILLE SYSTEM OF PAN CARS.

10 tons and having a lifting power of 4 tons. These several machines were of the same weight, and the efficiency of work was dependent upon the manner of deriving power from the engine in hoisting the weight. In the first machine a friction clutch was used, being easily thrown in and out of gear, the hoisting drum receiving the chain from the crane, and paid in and out at the control of the engineer. An endless-belt connection with the crank shaft, and running around the sprocket wheel on the car axle, furnished propulsion. The body of the crane rests on a central axis, allowing it to swing through a circle. The axis of the crane has a large gear wheel, into which fits a smaller gear wheel, receiving power from the crank-shaft. The mode of action is as follows : The rock, when broken up, is loaded by hand into wrought-iron cylindrical buckets, some holding ½ cubic meter and large sizes holding 1 cubic meter, the task men receiving twenty to thirty-five cents per cubic meter for this work. The buckets are made fast to the end of the chain and the engineer of the crane hoists them up and swings them around onto cars, where they are dumped. The capacity of work has varied largely, but at San Pablo and Bas Obispo they average nine cars a day of 6 cubic meters each; but this is much too small, as when properly supplied with buckets they might accomplish at least 100 cubic meters work. A limited number of men working in a small area could fill only as many buckets as space would allow; and meanwhile the crane might travel up or down the track to different squads of men, hoisting and emptying the buckets as fast as filled, the dump-cars standing on the second track, behind the track of crane, and in this manner would accomplish a much greater amount of work. In the rainy season, when construction tracks are difficult to keep up, this crane is especially useful, as, on account of its light weight and low center of gravity, it is not as easily upset as excavators, and it has been frequently used in earth work. The working force necessary is one engineer to each crane, paid $3 to $5 per day, and one fireman to three cranes, receiving $2 per day ; one carman and one bucketman to each crane, $1.50 each per day. The coal used per day averages ½ ton. In machines Nos. 2 and 3 the power is derived from a gear wheel on the crank-shaft connecting with the gear wheel of the hoisting drum. The boilers are horizontal and the center of gravity is thrown as far to rear of crane as possible by loading the platform on back with rails as heavy ballast. When ready for work the frames are in some cases "jacked up," giving a wider support for a working base,

as the arms are 8 feet centers and jacks 7 feet apart. Of the five varieties in use, No. 1 has executed good work, and the less liability of parts to get out of order is an important element. These machines are built in sections and transported and put up on the scene of work. Cost, F. O. B., about $5 000.

Ponton Biques.—Large floating cranes are used at Colon and Panama in trans-shipping a heavy cargo from deeply laden vessels of large draught, on account of their inability to run alongside of the canal wharves. The largest size has a lifting capacity of 40 000 pounds. The hull is 60 x 30 feet and of 10-feet draught, supporting a derrick on the forward end of 75 feet height and a reach over sides of hull of 30 feet. This crane has been used in lifting the heavy sections of dredges from the holds of ships, and swinging them around and loading them onto flat cars of the Panama Railroad for shipment upon the line. This crane has the power of a stationary crane, and may be moved from place to place when necessary, by a system of guys, which are reeled up on hoisting drums on board of the ponton. The crew necessary is, one engineer, one fireman, one man on the tower, two men in charge of guys, engineer in charge of hoisting engine, and engineer in charge of "swinging derrick." The use is limited at Colon to unloading vessels and it is seldom at work more than half of the time. On account of gaining the proper stability it must have an immense hull to support the weight. The entire plant is expensive in the extreme, and if a proper wharf had been first constructed, dredged out its entire length, and a track laid to allow the passage of freight cars, a permanent crane might have been erected at half the cost and less expense of operating.

Marine Cranes.—I noticed a number of self-propelling steam cranes at Colon, having a capacity of hull to hold 500 tons of freight. These cranes went alongside the steamers, made fast and discharged the cargo of the steamers, receiving it with the chain and crane, and swinging it around to the hull of crane until filled, when another crane would come alongside and repeat the operation. These cranes would then steam up the line and unload at the different shops or storehouses. In all cases they were used in unloading the heavy parts of machinery, such as boilers of dredges, excavators, cranes, locomotives and dump cars. Their capacity was 10 tons and usually two cranes were mounted on each hull. These floating cranes of this size I regard as indispensable, as at some time all available wharf room might

PLATE L.

FRENCH AND ENGLISH CRANES.

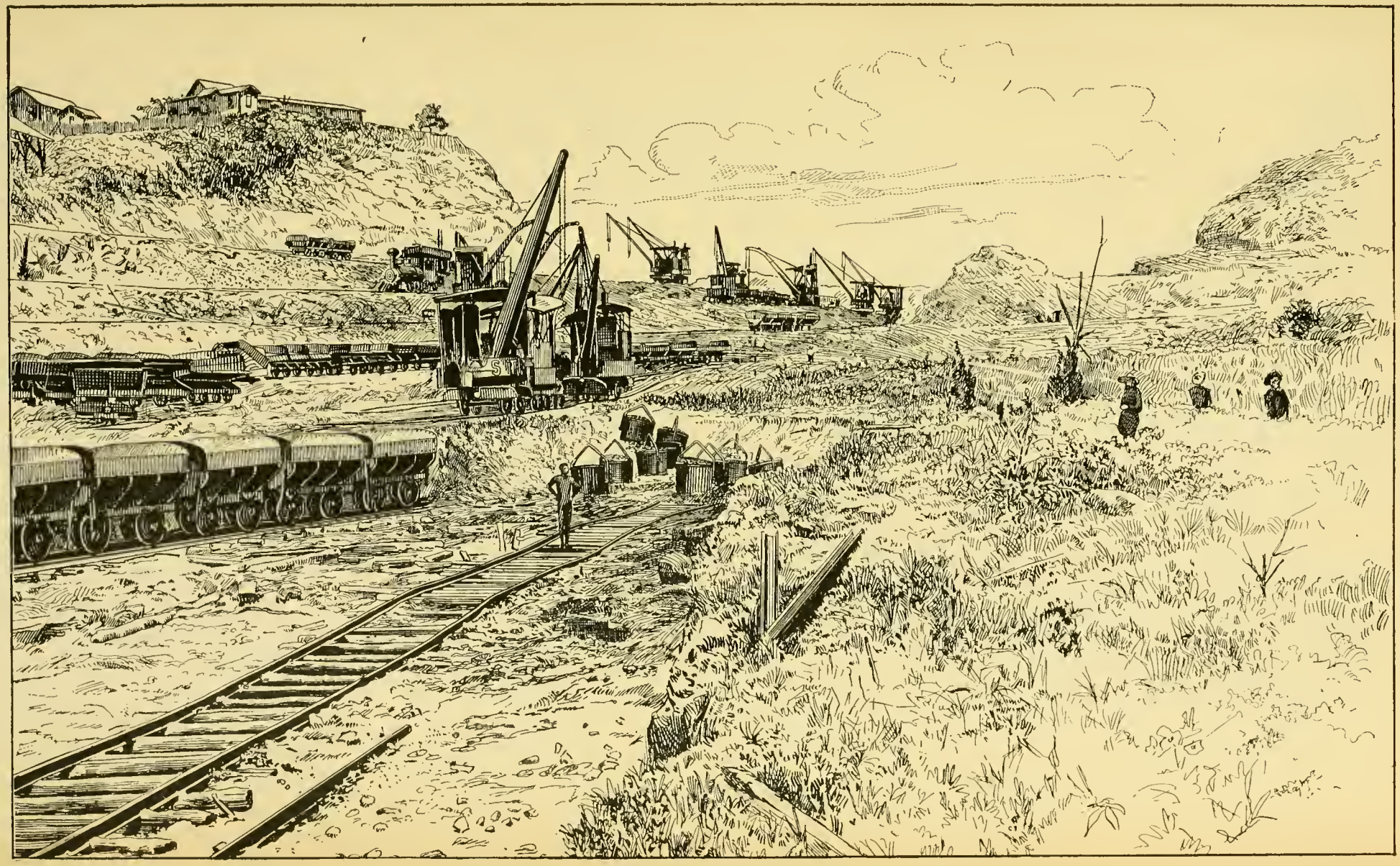


PLATE LI. CRANE AND BUCKET SYSTEM AT OBISPO.

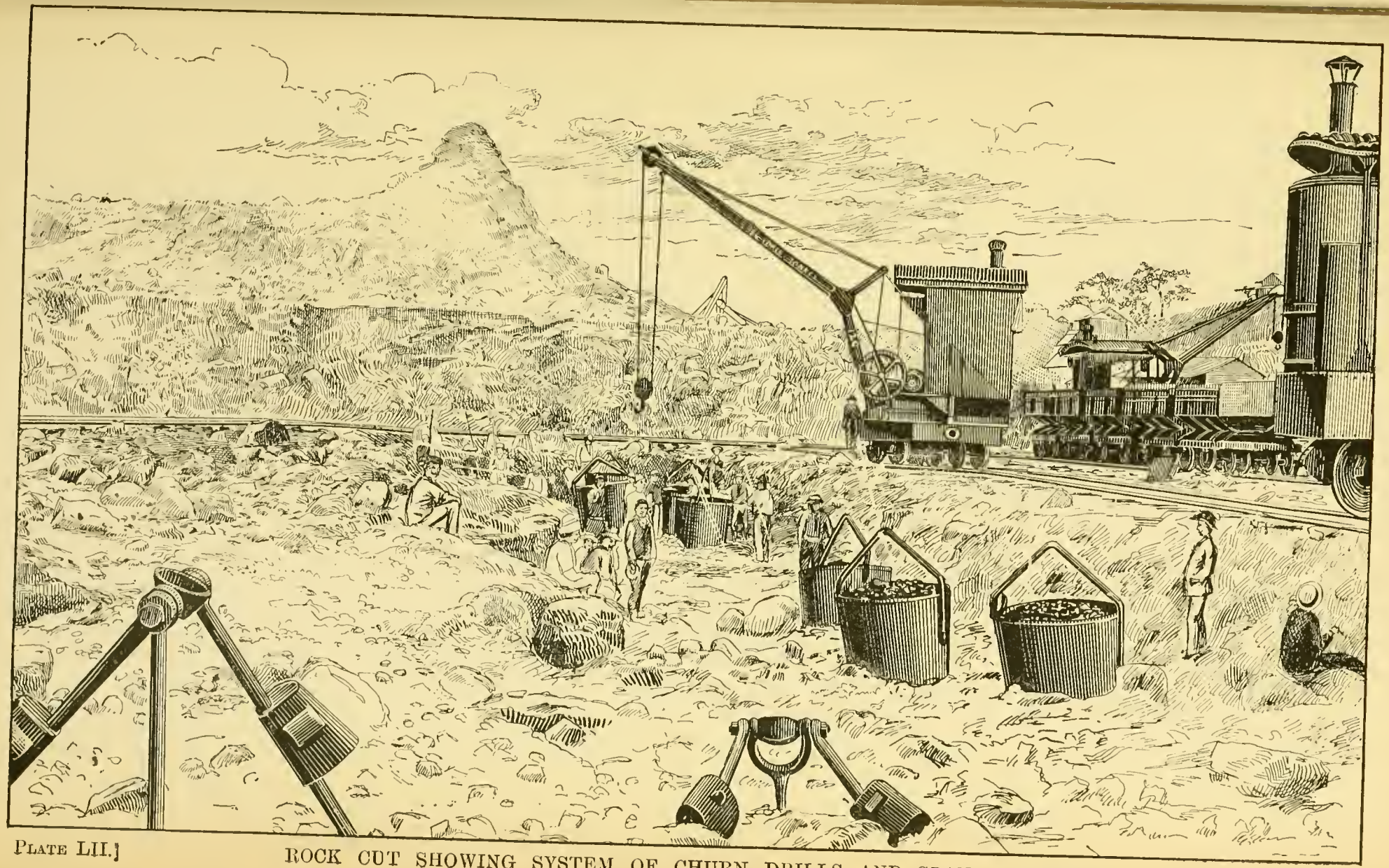


PLATE LII.] ROCK CUT SHOWING SYSTEM OF CHURN DRILLS AND CRANE BUCKETS.

be in use and a steamer arriving at this time would suffer demurrage in not being able to discharge her cargo, while a floating crane might make fast alongside when the steamer was anchored in the harbor, and discharge into the hull of crane, or upon scows alongside. Many steamers are provided with cranes or use tackles and yard arms, but it would be slow work as compared to a distinct plant for the purpose.

Excavators (See Plates LIII, LIV, LV, LVI, LVII).—Many styles of the endless-chain-of-bucket excavators are in use, but after a thorough examination as to machinery and work accomplished, I consider the machines constructed by P. DeMange and Messrs. Satre & Lyons superior to any others. This machine is constructed on two trucks of four wheels each and of a 5-foot gauge, while when in working condition the working base is broadened by jacking up and throwing the weight on the working side out to a third rail 7 feet distant from the line rail. In transporting the machine from place to place the construction track of 5 feet gauge is adequate, and in working a third rail is laid up and down the line over which the machine is to travel. The ladder over which the buckets travel is hung on an axis on the back of the machine, but throws the center of gravity towards the working side, and to offset this, ballast of railroad iron is loaded on the extension on the back of the machine. The boilers are usually horizontal, giving low center of gravity, and the water tank of iron and a coal bunker of iron are placed on the boiler end of the platform. There are two styles of attacking a bank, called "down diggers" and "up diggers," whether attacking material above or below level of track. A "down digger" of the above manufacture at work at Tavernilla may be described as follows (Plate LIII): The endless chain of buckets operated on "over and under" system received their motion from the square upper tumbler of the shaft, which was geared from its wheel of large circumference, connecting with a smaller gear wheel on the crank-shaft. The vertical engine supplied this shaft with 3 pistons, giving a positive motion. The buckets were constructed with a quadrangular hemispherical face and no back, the links being hung at the back of the bucket, and the ladder being suspended down the bank, was raised or lowered to give a slight contact to the cutting nose of the bucket, which became filled by gradually cutting a slice all of the way up the bank, carrying the material up and onto the upper tumbler shaft, and as the bucket at a slope of 45 degrees to the horizontal changed its angle by going over the upper tumbler, no sup-

port was given to the contents, which fell out and down into the hopper, and through a "chute" suspended over a train of dump cars on the second track and back of the machine.

The engineer of the excavator controls its movements entirely, raising and lowering the ladder, also moving the excavator up and down the track by an endless belt running from a sprocket wheel on the crank-shaft of the engine to a sprocket wheel on car axle. With good executive management and a constant supply of cars under the chute this machine could excavate 200 to 300 cars per day of 6 cubic yards each.

The force necessary to work is as follows:

1 engineer	$5 per day.
1 coalman	2 "
1 fireman	3.50 "
3 carmen	2 each per day.
1 chainman	2 per day.

In connection with the excavator would be needed two locomotives, with fifteen to twenty cars each, depending upon the distance of the dump from the centre of excavation and the grades to be overcome in traversing the distance. The consumption of coal is 3 tons per day. In sand and soft clay and dirt the machine will do efficient work, and in the Culebra Cut I noticed one of the machines doing slow work in broken rock. The stability of the machine is also controlled by clamping the frame of the machine on the rear to the track.

The Osgood Excavator. (See Plate LIV.)—I saw three of these machines at work at Culebra and Paraiso, which were amongst the original shipment of seven sent from America. The boilers were vertical, of 50 horse-power, cylinders 10 by 12 inches. Buckets holding 2 cubic yards, struck measurement. Cost was $8 000, F. O. B. at Albany, N. Y. The working force was as follows:

1 engineer	$4 to $5 per day.
1 dipper tender	2.50 to $3.50 per day.
1 fireman	1.75 per day.
6 men shifting truck, carrying water, etc	1.50 per day.
Consumption of coal	1½ to 2 tons.

This type of machine is supported on two trucks of 5-feet gauge, and when in position for working the forward truck of the machine is

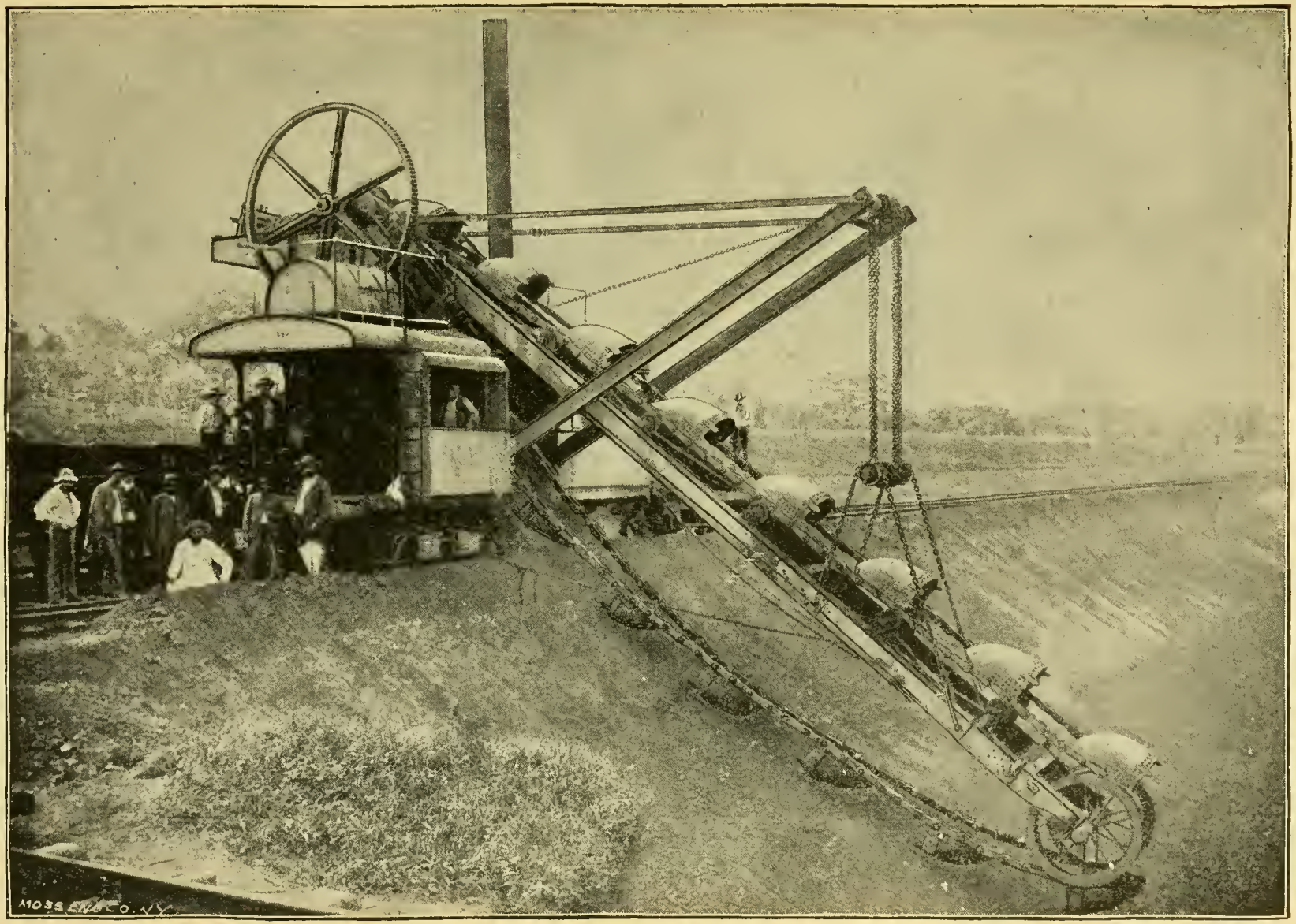


PLATE LIII. FRENCH EXCAVATOR, "DOWN DIGGER" TYPE.

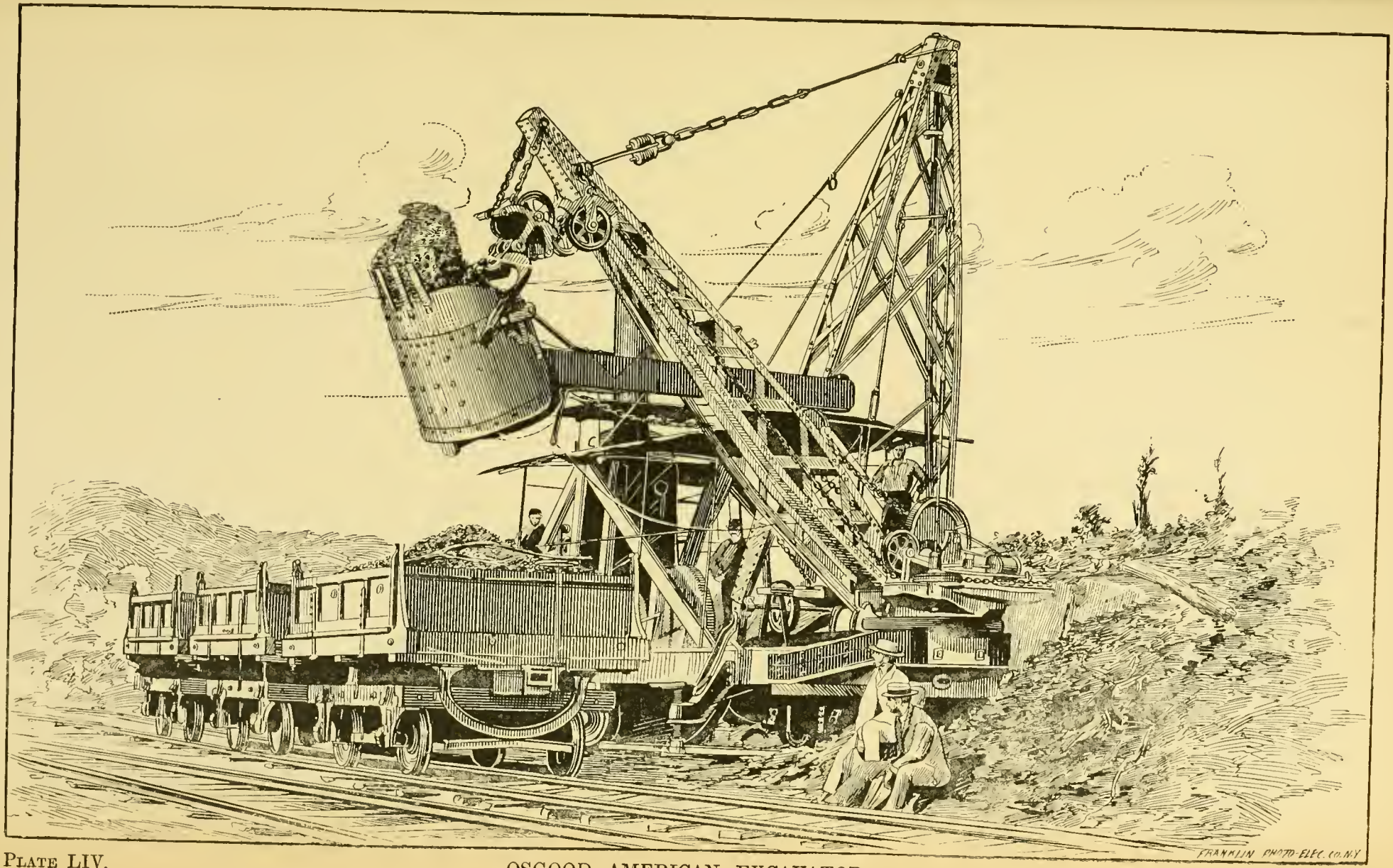


PLATE LIV.

OSGOOD AMERICAN EXCAVATOR.

jacked up, throwing the weight off the rails, and the outriggers of 8-feet centers are used instead, giving a wider working base. The weight of these large type of machines is about 30 tons. Self-propulsion is gained by an endless belt connection with the main engine shaft to the rear axles. Water tank and coal tank are placed on the rear car near the boiler. These machines will excavate a cut up to 70 feet in width, and dump contents of dipper 29 feet above track. The mode of action is for the excavator to start at face of cut and gradually excavate forward and on each side, of sufficient width for the placing of two tracks, one on each side of the excavator, which may move forward in reaches of 8 feet, each digging her own track. Dump cars are brought in alongside on either track from the rear switches by cable connection winding around a drum on the exterior of the body of the excavator; these cars when filled are hauled out onto main line, and empty cars are in readiness to supply their places. The dipper delivers first on one side, then on the other, the cars being constantly supplied. Under favorable conditions and management, in sand and loose gravel, as high as 2 000 yards per day of ten hours have been accomplished. In hard-pan and obdurate clay satisfactory work has been done. The engineer on the platform of the car operates the engine, lowering or raising the dipper, swinging it around onto the cars, while the dipper tender standing on the turn-table crowds the dipper to its work and empties it. I found in all cases the "Osgood" machine spoken of in the highest terms, as doing a larger amount of work with less force of men and less liability to get out of order than those of foreign manufacture, and the fact that it was able to help itself, preparing its own track, enables it to work in all places where the endless-chain-of-bucket type would be useless.

Excavator "Up-digger." (See Plate LV.)—These machines are constructed receiving their power in two ways from the main engine, by direct connection of piston to crank-shaft, on which are balance wheels, and two small gear wheels gearing into wheels of large diameter on upper tumbler shaft. This supplies a positive power, so important for a machine at work in stiff clay and hard-pan. The other means of doing this is applied in the Suez Canal type of excavator, which receives power by the sprocket-and-chain connection already described as used on the French dredges.

In the "up-diggers" the buckets are constructed similar to buckets of the dredges, and have the under-and-over movement.

The weight of these machines runs as high as 36 tons ready for work, and to this must be added the weight of the dirt of one-half the number of buckets filled, so in many cases the weight will reach 50 tons. In the larger type a third rail is placed 7 feet from the rail on the "line side," thus throwing the weight of ladder and derrick on the outer rail, while clamps on the line side are easily adjusted on the track and body of the excavator, preventing any overturning. The "up-diggers" are limited as to the amount of work they are to execute, on account of the short length of ladder in use, and the track must be constantly "thrown," in order to give a steady working contact with the bank. In working "down-diggers" and "up-diggers," a supply of cars should always be on hand, so that when one train is loaded and starts off for the dump, its place may be taken immediately by a second train. This item is alone important in the capacity of work of a machine, and a foreman of intelligence should always be placed in charge, so to regulate switches and side tracks as to cause no demurrage. In the rainy season it has been found impossible to keep up the tracks at Culebra and Tavernilla, and the excavators have all been abandoned and cranes substituted. The "down-digger" machines rest on the edge of the terrace, and the slope, inclined at an angle of two to one, is not flat enough to prevent a sliding tendency of the bank caused by a 50-ton weight, and a vibratory movement arising from the contact of the bucket with the bank. With the "up-digger" at work in the rainy season the same sliding occurs, the limited swing of ladder (in many cases the working side of machine being only 10 feet from the perpendicular bank, which caves and slides) causes the tracks to be covered up.

Excavators and Transporters.—At Tavernilla, in connection with the endless-chain-of-bucket excavator, was employed a transporter (Plate LVI). A truss of 500 feet in length was supported at one end on the deck of the excavator, and extended at right angles to the fore and aft line of work, the other end being supported upon a traveling derrick and car. This truss was 6 feet in width and 10 feet in height, and an endless belt 4½ feet wide received motion from an independent engine of the excavator. The contents of the buckets of the excavator were discharged into the hopper and out onto this traveling belt, thence along over top of truss until the end was reached, when, the belt going over the outer tumbler, its contents fell to the ground. The outer end of the truss may be raised as high as 30 feet from the

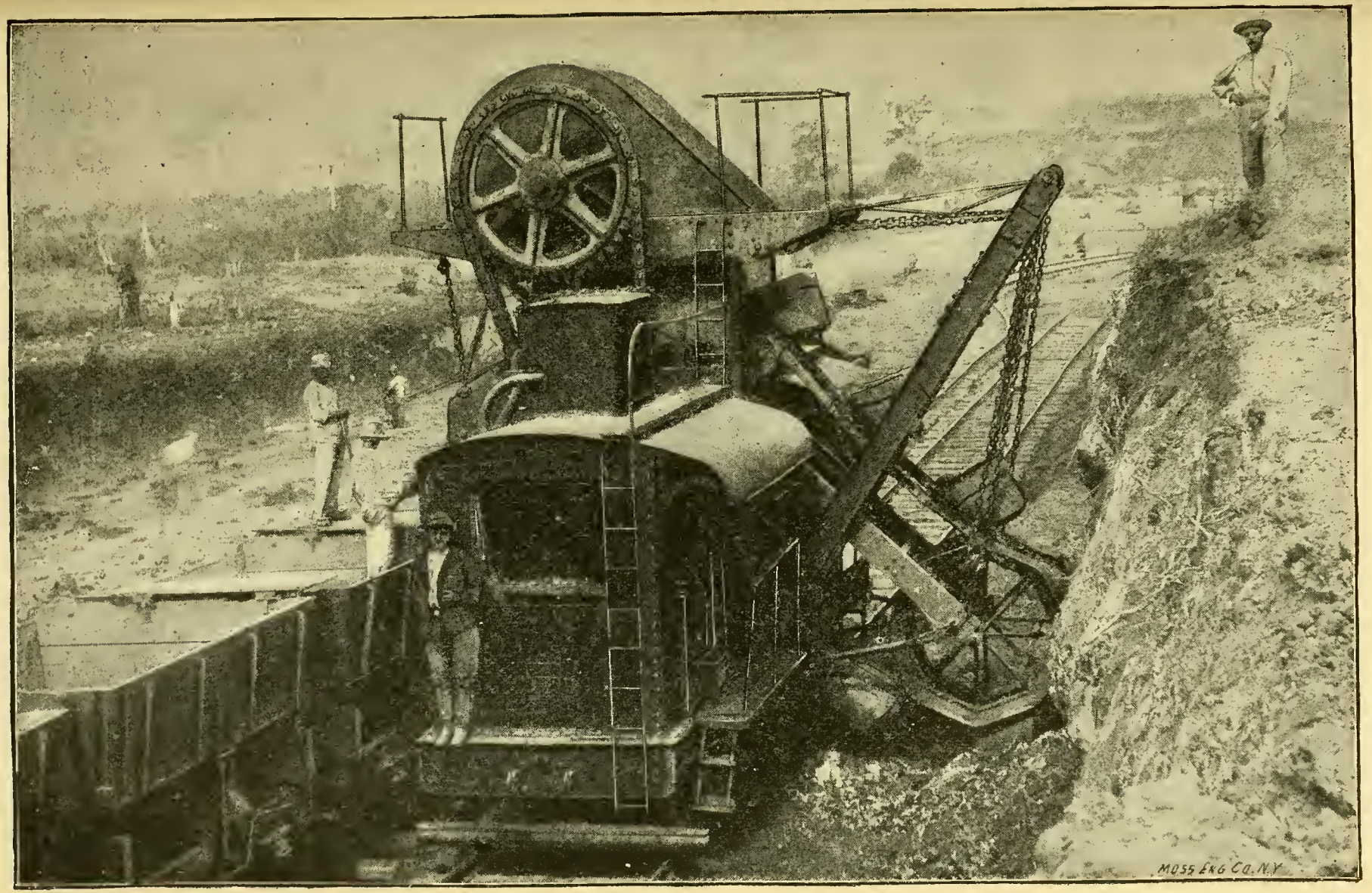


PLATE LV. FRENCH EXCAVATOR, "UP DIGGER" TYPE

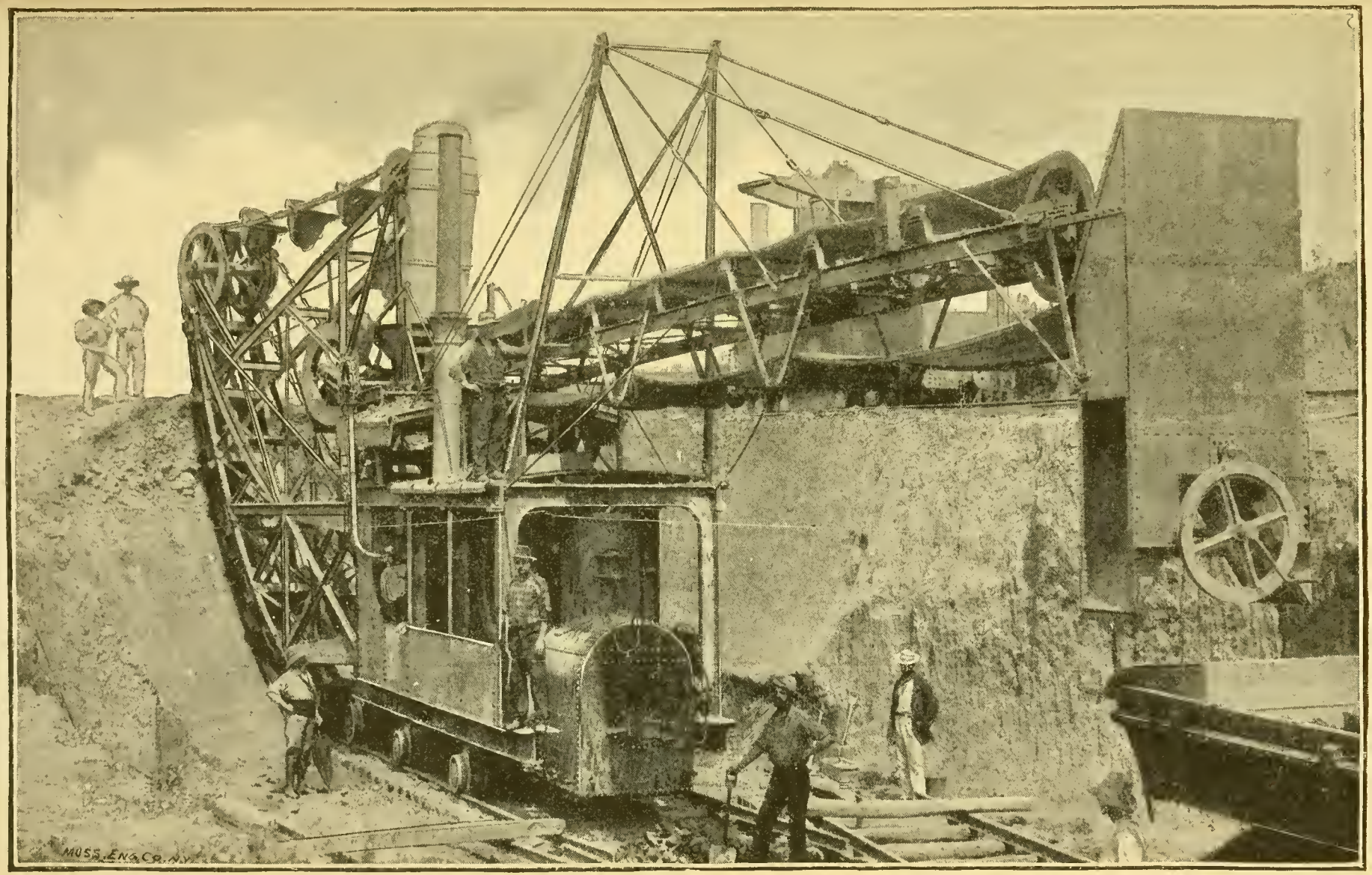


PLATE LVI. EXCAVATOR AND TRANSPORTER.

PLATE LVII. PARAISO SECTION, FRENCH AND OSGOOD EXCAVATORS.

ground, giving room for a large bank to fall without obstructing the passage of derrick car from the débris sliding toward the machine. In this work the excavator was digging 30 feet below the rail of the car, material was carried 500 feet distant and elevated a total of 50 feet. This necessitated the keeping up of three tracks, the two tracks of excavator, and a track of platform car, supporting belt, engine and boiler, and track of derrick car. In addition to the regular crew of the excavator there were used one man on the derrick car to preserve a constant forward motion up the track with the excavator, one engineer on the platform car to regulate belt engine, and one fireman each for derrick car and platform car. The use of this extensive plant is limited to the following case: In deep cuttings, where a depth of over 50 feet is to be made and over a length of several hundred feet, it is necessary to throw the material so far back from the center as to be outside of the slopes when the requisite depth is obtained. In this first cutting of 30 feet, throughout width and length, the material may be handled automatically and deposited by the transporter, but here its usefulness ends. In the second cutting of 30 feet the material must be hauled away by cars, and tracks constructed up and out of the cut, and to some dumping ground, as in going down deeper with the excavation the dump rises vertically, and the transporter would be at too great an incline to work economically.

Clapets.—The self-dumping steam clapets were all of French construction, the hulls in many cases being constructed in sections in France and sent over to the Isthmus and set up at the foundries and shops at La Boca and Colon. The capacity varied in instances from 150 to 300 cubic meters, the length of hull being from 75 to 120 feet. Propulsion was from the aid of twin screws, and the draft of water when loaded 10 feet and under. These machines were supplied with hinged doors on the bottom of the scow, opening outward, and were opened and closed by chains on the hoisting engines. They are much easier handled than self-dumping scows operated with tow-boats, and the time occupied in running up to the dredge and making fast is reduced to a minimum, usually attaching and detaching in ten minutes. The cost of these clapets was as high as $20 000 constructed on the Isthmus. The complement of men was as follows:

1	Captain	$200 per month.
1	Mate	120 "
1	Engineer	120 "
1	Cook	60 "
2	Firemen	50 "
10	Deck-hands	45 "
1	Donkey engineer	45 "

The contractor supplying board.

Houses for Men.—The common type of quarters for men along the line of the Panama Canal was to erect a light frame-work of wood, the sheathing being horizontal and of spruce, the roof of corrugated iron, and either round or peaked. Stationary beds were erected for the men through the center of the house in rows of two deep, and houses of 50 by 30 feet will accommodate 40 men. The foundation was constructed of rough ashlar masonry or concrete in piers located under the corners, quarters and centers of the floor, in most cases the house being raised high enough to allow free circulation of the air underneath. "Flying Houses" are frames built of light angle iron with corrugated iron roof and sidings, and are largely used for machine shops, when the use will be temporary. They can be easily taken apart and transported from place to place by removing the bolts holding the sections, while with wooden structures, nails and bolts being used, the timbers would become broken, split and decay in a short time. Tile roofs are much cooler than corrugated iron, but more expensive, while shingles decay in a year's usage. Asbestos and gravel roofing have not stood repeated tests, corrugated iron having given the best satisfaction.

Dump Cars—Everard System. (See Plate LVIII.)—I saw many wooden systems of dump cars, built in some cases of creosoted wood in order to resist the action of the climate, but I found upon examination that all cars so constructed did not last as long as those built of wrought-iron. The body of the car being subjected to the concussions of heavy weights falling from the hopper of excavators soon became fractured. These cars were constructed of 5-feet gauge and four wheeled, the axle and frame-work and body of cars being easily taken apart and occupying small space. There were two sizes of bodies, one holding 4 cubic meters and one holding 6 cubic meters. The weight of the larger car was 5 tons unloaded, and when loaded with 6 cubic meters

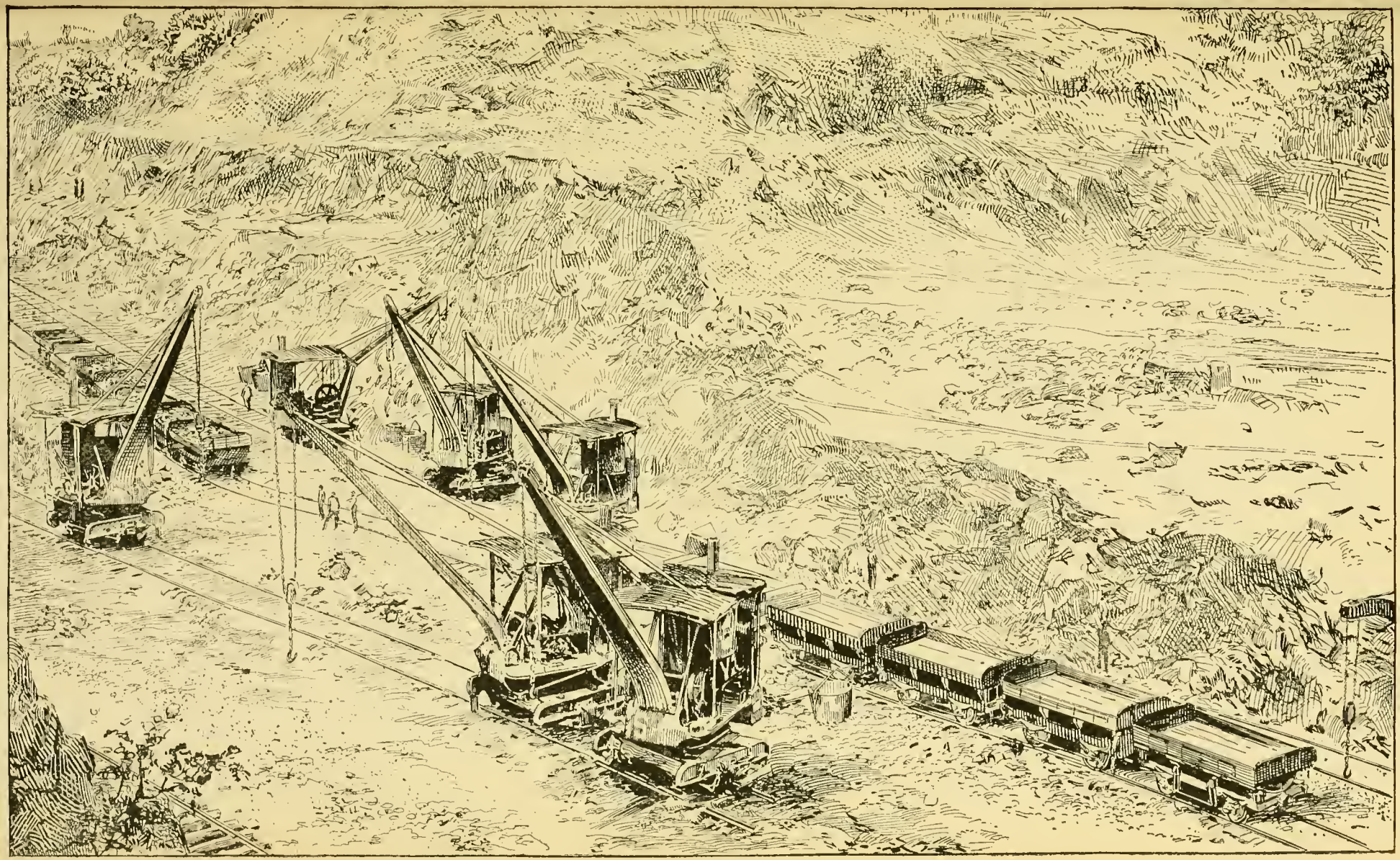


PLATE LVIII. ROCK CUT AT OBISPO.—EVERARD DUMP CARS.

of earth, 10 tons. When the car is evenly loaded the center of gravity is well preserved and the car may be easily dumped by the aid of two men. At each end of the body is a hemispherical cam, resting vertically upon a track supported upon the frame of the car, so that the body is allowed to move through an arc of 45 degrees. One serious objection which may be easily remedied is the use of a buffer in one continuous piece, running underneath the car and connected by a lug with a spiral spring. If cars are run into, the concussion causes the buffer to retreat, and the motion is not sufficiently counteracted by the spring, it being too light, and consequently the buffer from the shock is knocked out of place, retreats to rear, and striking the buffer of the second car a buckling occurs.

I would suggest that this spring be doubled in strength and the buffer then would be enabled to stand the shock of adjoining cars. The handles used to keep the car in a horizontal position are much too long and often strike against the dumps along the line when not leveled off. Shorter handles would be more effective, as in a car upset on the dump no more leverage is gained in using a long handle to restore it to a normal position than by using a short handle, which is more rigid and admits of no lateral pull.

I have not attempted to explain in detail the relative merits of the different machines, but have only described the plant doing the most effective work, and later in the year I hope to present a paper on the "Dredging Appliances" in use on the Panama Line, with detail drawings of each and a history of their work.

The same machinery in use on the Panama Line, when worked by intelligent Americans and operated by American contractors, would accomplish a larger ratio of work; but this will never occur, as the climate alone forbids the economical employment of any number of skilled white men.

In connection with the paper on the "Plant and Machinery on the Panama Canal," it may be interesting to engineers to be informed how much work has been accomplished by the use of this plant on the canal, and how much work remains to be done, and the probable cost of that work.

The first estimate of the work to complete a sea-level canal, stated at the "International Congress" at Paris in 1879, was a cube of 46 000 000 meters, which was increased later to 75 000 000 cubic meters and 105-000 000 cubic meters; while in 1885 this estimate was still further increased to 151 000 000 cubic meters, of which 20 000 000 cubic meters was for "derivations."

The official reports state that up to June, 1888, the Canal Company had expended $177 910 000 and had accomplished 49 000 000 cubic meters, showing an expenditure of $3 62 per meter. This work has been accomplished under conditions much more favorable than will be encountered in the future, when, as the cuts are deepened and the material becomes harder, the length of haul and time of handling will increase the cost of the work materially.

The total par value of the several loans to June, 1888, is $351 150 900, the loans having first been taken in 1882 at a discount of 12½ per cent., bearing interest at 5 per cent., while the last loan was floated at par, bearing 3 per cent. interest, and was finally taken up at 56 per cent. discount.

The interest and fixed charges on this amount expended is more than $20 000 000 per annum.

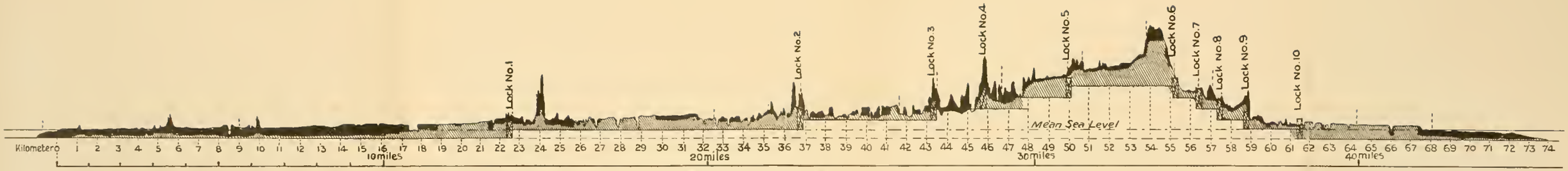
An accurate idea can be gained by looking at the profile on Plate LIX.

This profile was issued by the Canal Company in Paris in January, 1888, for the purpose of infusing new interest amongst financiers in the adoption of the lock system. The showing of the profile and the comparison between work to be done and work already accomplished was so dangerous to the Canal Company that the issue of this profile was suppressed.

The first announcement made by the Canal Company that 40 000 000 cubic meters remained to be extracted under the lock system was still further reduced in the publication on May 2d in the "Bulletin du Canal Interoceanique," that only 32 132 244 cubic meters remained to be extracted.

A careful calculation by scale, as shown on the profile of the amount of excavation for the lock canal between kilometers 18 and 68, gives 37 250 627 cubic yards. These estimates are made by taking the center heights and slopes at 1½ to 1.

In a number of cases the center of the canal has been excavated deeper than the sides, so on this profile, which was taken along the cen-



PANAMA CANAL. PROGRESS PROFILE, JANUARY, 1888, WITH REFERENCE TO LOCKS.

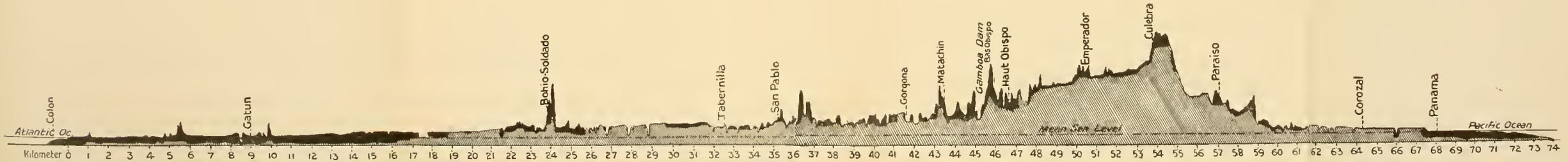

Colon
Atlantic Oc.
Gatun
Bohio-Soldado
Tabernilla
San Pablo
Gorgona
Matachin
Gamboa Dam
Bas Obispo
Haut Obispo
Emperador
Culebra
Paraiso
Corozal
Panama
Pacific Ocean
Mean Sea Level
Kilometers 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74

PLATE LIX.

PANAMA CANAL. PROGRESS PROFILE, JANUARY, 1888, WITH REFERENCE TO SEA LEVEL.

ter line, we do not get the average cutting, but a height much more favorable to the canal than against it. The slopes in many instances have been carried further back at an angle of 2 to 1 on account of the sliding tendency of the soil.

This estimate does not include the amount of excavation necessary for the "derivation" on either side of the canal, as while the location of the canal itself is made with a view to the most advantageous operation, it is not always possible to find equally favorable locations for the parallel channels.

In places the amount of excavation in the derivation channel exceeds the cube of the canal. Again, this aggregate amount does not include the excavation in deepening the canal from 15 to 27 feet between kilometers 1 to 18, and the excavation from kilometer 68 to 74, which is of the most difficult nature, as coral rock has been met with all along this section.

It is safe to estimate that the excavation necessary to complete the lock canal and derivations will exceed 60 000 000 cubic meters, and the cost of handling $2 50 per cubic meter, requiring $150 000 000 in cash outlay.

The interest and fixed charges on work already accomplished for the next four years, the earliest possible time at which it might be completed, will be $80 000 000.

From past experience at least $2.50 for $1 will have to be paid for the negotiation of the new securities. This will necessitate obligations to be issued of $575 000 000, which, in addition to the $351 000 000 debt already incurred, would give a total of $926 000 000 as the cost of the completed canal.

The tonnage of the world availing itself of the use of the canal in 1892 will not be over 5 000 000 to 6 000 000, which, if taxed as high as $3 per ton, would only yield $18 000 000.

Then the cost of operating a lock canal with an artificial water supply will at least require the expenditure of $3 000 000 per year.

Even at this high rate of tax on tonnage, the canal would only yield a dividend of a little over two per cent. per year.

The future work and completion of the Panama project is purely theoretical, and problematical in the extreme, standing, as it does to-day, on the verge of a financial crisis, which may be hastened at any moment by De Lesseps' death, as no man has succeeded in raising such

sums of money for an enterprise which at the start was so misjudged and underestimated.

At this late date problems relating to the Chagres River control are still unsolved, and the blunders of the past will surely creep in during future work.

Now, the company's rate of excavation, even with the help of the large figures from the Colon—Gatun section, has not averaged 1 000 000 cubic meters per month, the past two years through. At the best rate of progress, therefore, and with the most liberal allowances, there is work enough to consume four years, and that would involve the disbursement of something like $80 000 000 for interest and sinking fund of the debt and expenses of administration, without counting a dollar for work or material on the canal itself.

Though the work accomplished thus far has been the easiest that the line presents, still, more than one contractor has succumbed before the difficulties of the Culebra. Some of the richest and most experienced firms of Europe have declared the tasks of that portion of the line beyond their power to accomplish, and have abandoned their contracts. Those tasks have not been lightened nor the difficulties lessened by the operations hitherto effected. The laborers are not more numerous nor more efficient than they were when Señor Tanco Armero declared that the results of their work convinced him that there had never been more than 5 000 men employed, though the company claimed to have 15 000.

Again, the water for the supply of the lock system, so far as is known, is to be furnished from the Chagres River. In the dry season the flow of the Chagres is at the rate of 10 cubic meters per second, of the Obispo 1 cubic meter, and of the Rio Grande 0.4 cubic meters. The three streams would therefore supply some 259 977 600 gallons per day. The capacity of each of the locks is 40 000 cubic meters, and each vessel crossing the summit level will require the two locks to be emptied once, using something like 80 000 cubic meters of water. The Panama Company claims a tonnage of 10 000 000 per year, or 28 000 tons per day, representing twenty vessels of 1 400 tons each, although this seems to us a most exaggerated estimate. The water required for their transit would be 322 400 000 gallons per day. The evaporation and filtration, estimated at one per cent. per day, would entail a loss of something like 47 300 000 gallons, making the daily requirement amount to 369 700 000 gallons. The supply is 259 977 600, and the shortage on this basis 109 765 260 gallons.

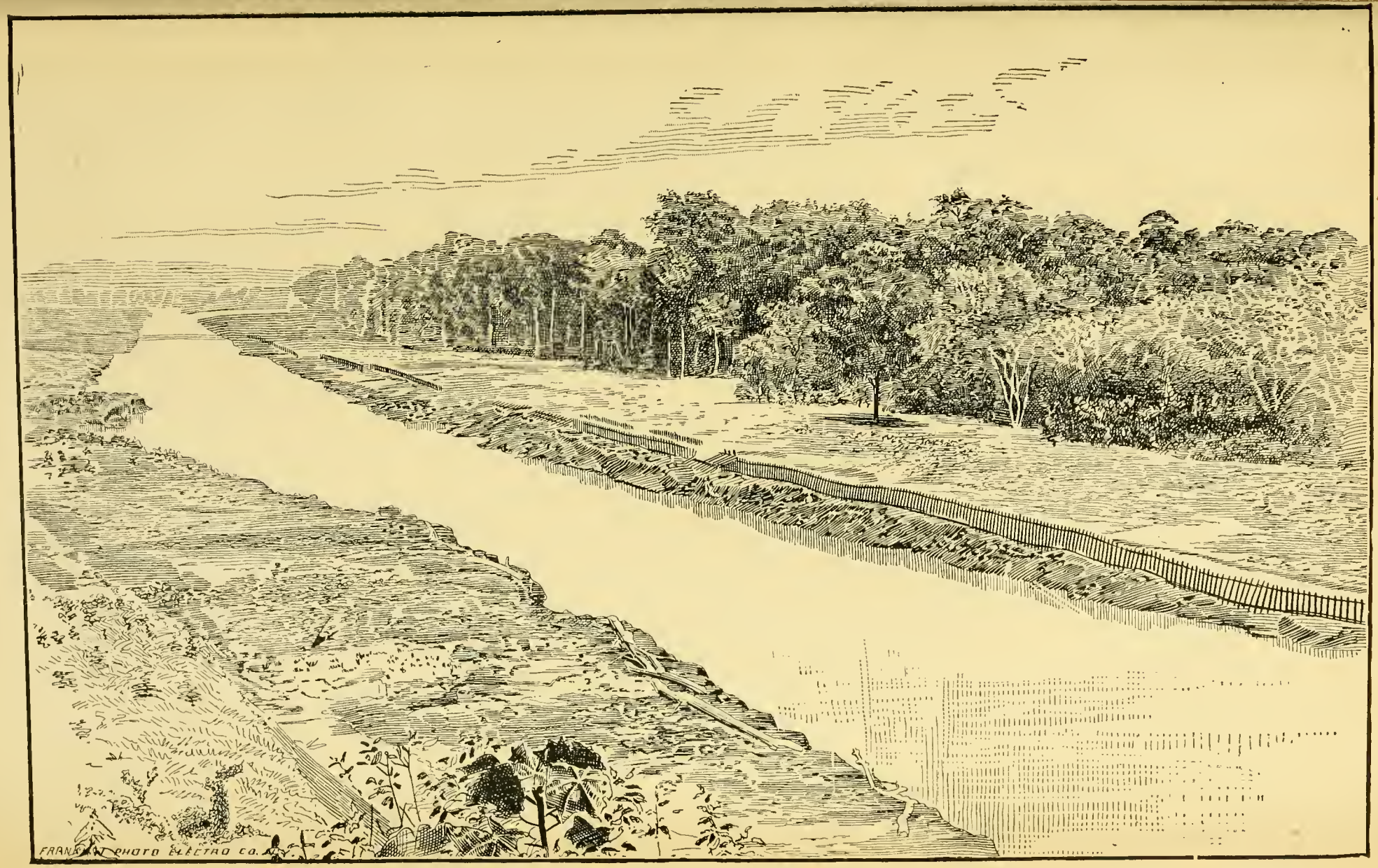


PLATE LX. SECTION OF CANAL, COLON SIDE, EXCAVATED IN LOW GROUND BY AMERICAN "HERCULES DREDGES.

In the rainy months the opposite conditions will prevail.

The Gamboa Dam, as originally proposed by French engineers, was to be 975 feet long on the base and 113 feet high, with outside slopes of 4 to 1, and was to consist of 10 000 000 cubic meters of rock and clay. The site was to be dredged some 60 feet to a solid foundation. These plans have been changed on account of the adoption of the lock system and the dam will only be carried to a height of 95 feet. Its total contents are 3 000 000 cubic meters, and already about 30 000 cubic meters have been deposited on the flanks of the hills at the two ends. No excavation has been attempted to gain a solid foundation, the theory being that on account of the immense weight of the deposited material it will gradually settle to a compact foundation. The original plans contemplated a receiving reservoir holding 1 215 000 000 cubic meters, which would be adequate to hold the waters of a six-day flood, the longest continuous rain-fall of the past ten years. By the new plan the reservoir will only hold 160 000 000 cubic meters, and the maximum discharge of the Chagres will fill it in twenty-seven hours if empty; but in the rainy season the dam will be well filled, and in case of an extended rain-fall the dam will only be available for storing water for pumping purposes for filling of the upper summit level, while the unruly Chagres will still be uncontrolled.

From kilometers 45 to 23 the adjoining country is drained by the Chagres River, and in the event of a six-days' flood its rise becomes enormous. In some cases the Chagres flowing in its original channel approaches within a few feet of the canal, when it is suddenly turned into a derivation channel, in some cases at right angles to its former course. Unless walls of masonry are constructed to protect these banks from the eddying effects of the stream, the Chagres will break through these mud banks and destroy the canal.

At many points between Gamboa and Colon, during the rainy season, the bed of the canal is lower than the Chagres, and is overflowed by it, completely stopping work on those sections. The sudden breaks in the surface shown on the profile are all crossings of the Chagres. Hence the small results shown for seven years' operations in this portion of the line; and it is apparent that until the deviations for the river on both sides of the canal are adequate to control the maximum drainage the work on these lower sections cannot make substantial progress. One of the objects of the construction of the Gamboa dam is to control

the flow of the Chagres, so that this part of the canal may be protected from its floods; but the cube of the deviations which have so far been made or proposed is only about one-quarter of the capacity required by the river in the height of the rainy season, the constructing engineers having adopted the view that the river will gradually enlarge the capacity of the deviations to its own requirements.

In April last I steamed from Colon up to kilometer 15, this length of canal being located in flat and marshy land, and the work has been accomplished by the Hercules dredges in discharging the material on either bank; but this work can only be carried on in the upper levels at enormous expense and trouble, as is shown by the lack of progress beyond kilometer 15 on the profile.

This open section of the Panama Canal (Plate LX) has been the subject of much comment and congratulation on the part of interested persons, both here and in France. The contractors have used it to show the efficiency of their work, and the Canal Company's supporters have printed pictures of it, and elaborated its advantages for developing local trade and for use in transporting material and supplies to the works. In view of the uncontrolled flow of the Chagres, and the liability of this lower portion of the canal to damage from caving and silting, the policy of dredging it years before the whole can be completed, so as to render this part of use, would seem, to put it mildly, very questionable. The monthly cube of extraction reported at Paris has undoubtedly benefited largely by it, and pictures and descriptions of "sixteen finished kilometers of sea-level canal" may have been of value at certain critical moments; but if the "open section" is ever to form part of an interoceanic canal it will probably have to be all dredged out again some years hence, and the interest on the money paid for it would make a comfortable sum, which has been completely thrown away. It will be apparent to engineers that with the rest of the work in the state shown, this section should still be lying untouched, if the work as a whole was to be prosecuted economically, without regard to the necessity of impressing the gullible and ignorant.

The Great Culebra Cut (Plate LXI).—This view, taken in April, 1888, shows the summit of the canal in the center of the Culebra cut, at kilometer 54, and exhibits the maximum amount of excavation done at any point on the line. The depth from the original surface to the bed of

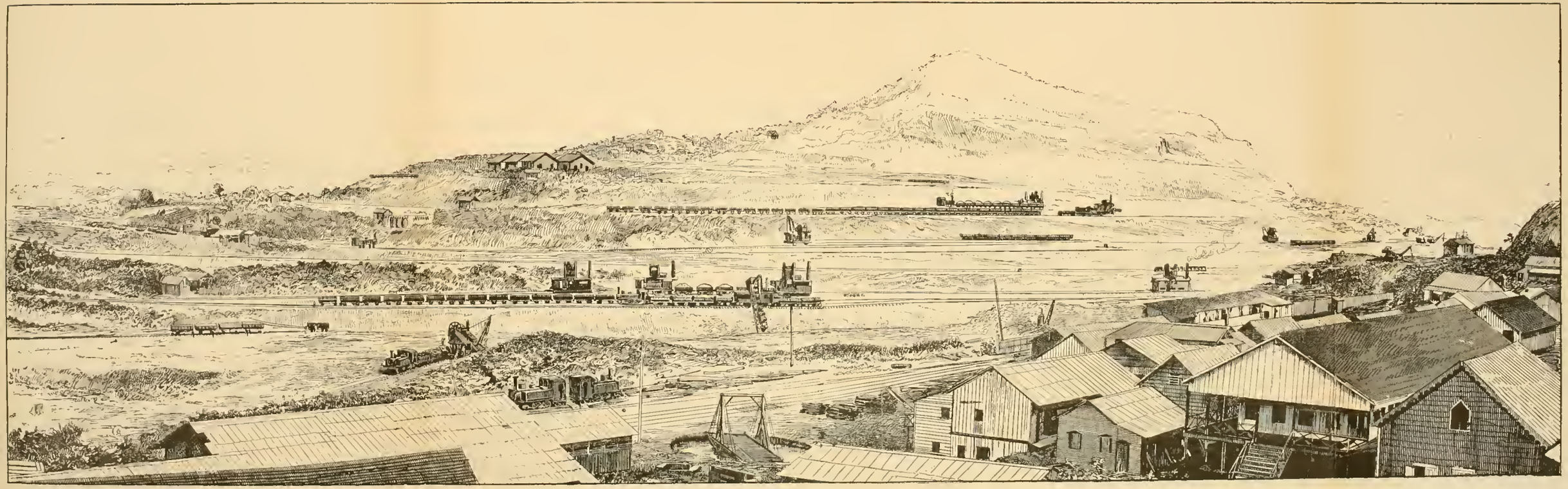


PLATE LXI. CULEBRA CUT.

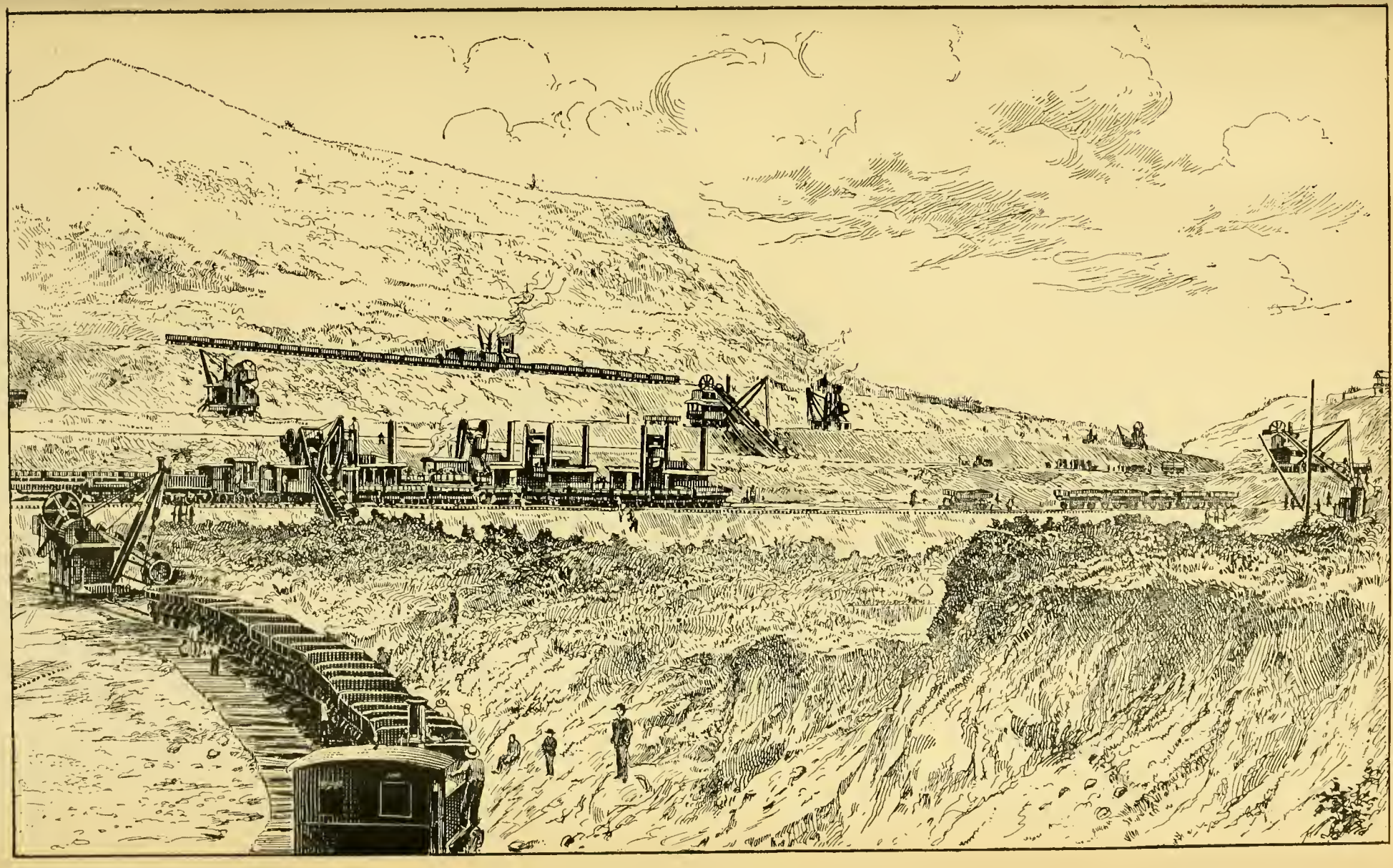


PLATE LXII. CULEBRA CUT, PANAMA SIDE.

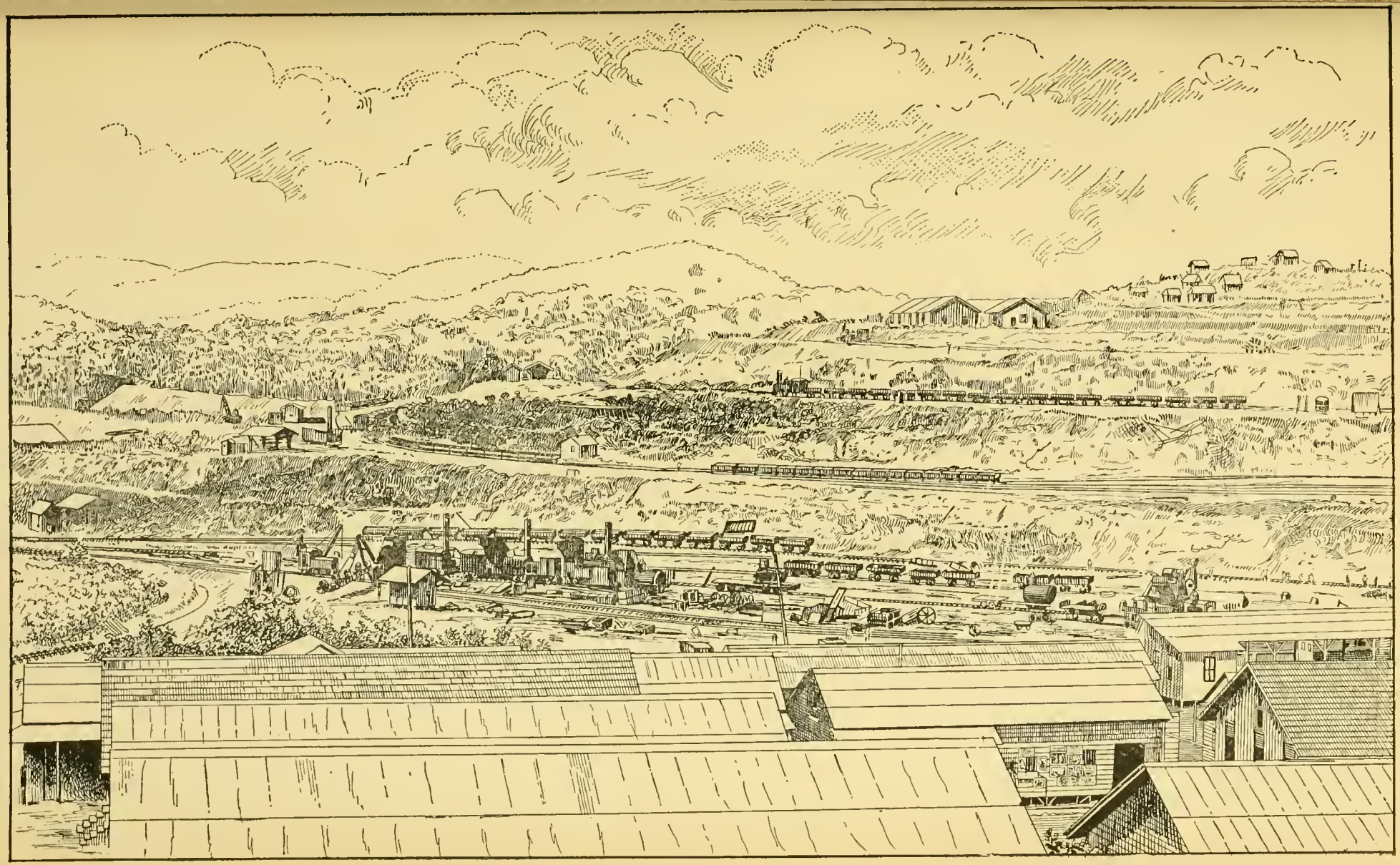


PLATE LXIII. CULEBRA CUT, COLON SIDE.

the sea-level canal at this point was 354 feet. About 93 feet have been taken out, leaving 111 feet to be excavated to the bed of the upper lock-level, as shown in the profile, or 261 feet to the bed of the sea-level canal, through a length of 3 300 feet. The abandonment of the upper lock would add 30 feet in depth for a distance of $3\frac{1}{8}$ miles; making the maximum cut 141 feet to the bed of the next lock-level, and the excavation for the lock canal through the central mass between Paraiso and Haut Obispo, a distance of $5\frac{1}{8}$ miles, would vary from 90 to 120 feet, apart from the 3 300 feet of the maximum cut. The profile shows at a glance that in general very little progress has been made in these difficult sections.

The surface width for a depth of 204 feet from the original surface to the bed of the upper lock-level, at a slope of $1\frac{1}{2}$ to 1, would be 750 feet. At this slope, which is the one now adopted, great difficulty is experienced from land slides, one of which may be seen in the upper center of the picture, on the face of the mountain. In consequence of this caving tendency the terraces have already been forced backward and upward on the mountain side, which has necessitated the readjustment of the lower terraces, and this characteristic will render further widening necessary as the cut is deepened.

In the earlier excavation it was possible to dump the material on both flanks of the mountain, upon the same level, and about half a mile away; but on the lower terraces the haul is largely increased, the dumping grounds being outside the cutting, on the extreme right and left, and requiring long trestles, 50 and 60 feet high, reaching out over the adjacent swamps that are being rapidly filled up. As the excavation descends the length of haul from the center of the works increases in a heavy ratio.

On the extreme left of the picture, in the lowest cutting in the foreground, the depth remaining to be excavated is over 100 feet to the lock level, or 250 to the sea level. In proportion with the deepening of the cut the hauling engines must overcome a heavy grade over an increasing vertical distance, and as the dumping grounds fill up, new ones must be found farther away.

It will be apparent that the excavation on the upper levels was accomplished with a minimum of labor and expense, and that as the cut descends the work will entail a rapidly increasing consumption of time and money in its execution.

The excavators chiefly employed here are of the endless-chain-of-buckets type, one or two of which are seen at work in the picture.

These were satisfactory in working upon the sandy material of the Isthmus of Suez, but they prove much less effective when attacking the hard pan and stiff clay of the Culebra. In connection with this plant are used Belgian 36-ton locomotives, and the Everard iron dump cars, as seen upon the railway tracks in the picture, which was taken while work was going on at the usual rate, as we understand, and will enable those familiar with public works to appreciate how likely it is that the remaining work will be completed by 1890 or 1891, when this is what has been accomplished in the seven years, 1881–88.

Plates LXII and LXIII are from photographs of other portions of the Culebra Cut.

The Emperador Cut. (Plates LXIV, LXV).—This view, taken in 1888, shows the canal line at kilometer 50.500, in the Emperador section, at the other end from Culebra of the great central level. The view is taken looking toward the Atlantic through the Valley of Emperador, in the direction of Haut Obispo. On the extreme left may be seen the dumping grounds for the excavations, reached by rail, and already used to a large extent. In the left center are the houses and shops of the contractors and the huts for the laborers.

The work done thus far amounts to little more than the removal of the surface earth. The excavation still required along this immediate front for two miles is 60 feet deep to the lock level, over 210 feet to the sea level. A distance of a mile and a half, to kilometer 53, requires from 90 to 120 feet of excavation to the lock level, or 240 to 270 to the sea level, while at kilometer 47 the excavation is 15 feet below the level of Lock No. 4, necessitating embankments, as is also the case at Lock No. 3, at Gamboa.

The deviation of the Rio Grande is unfinished, rendering progress exceedingly difficult in the rainy season, as the river overflows the works. There was erected here a through truss iron bridge to carry the Rio Grande over the canal and unite it with the Rio Obispo on the opposite bank, but on account of the change from the sea level to the lock system orders were sent from Paris in April to have the bridge taken down. This has been done, and it is stated at an expense greater than the original cost of the erection, and the parts are stored away to await further developments.

All the views given with this paper are reproductions of photographs actually taken upon the ground.

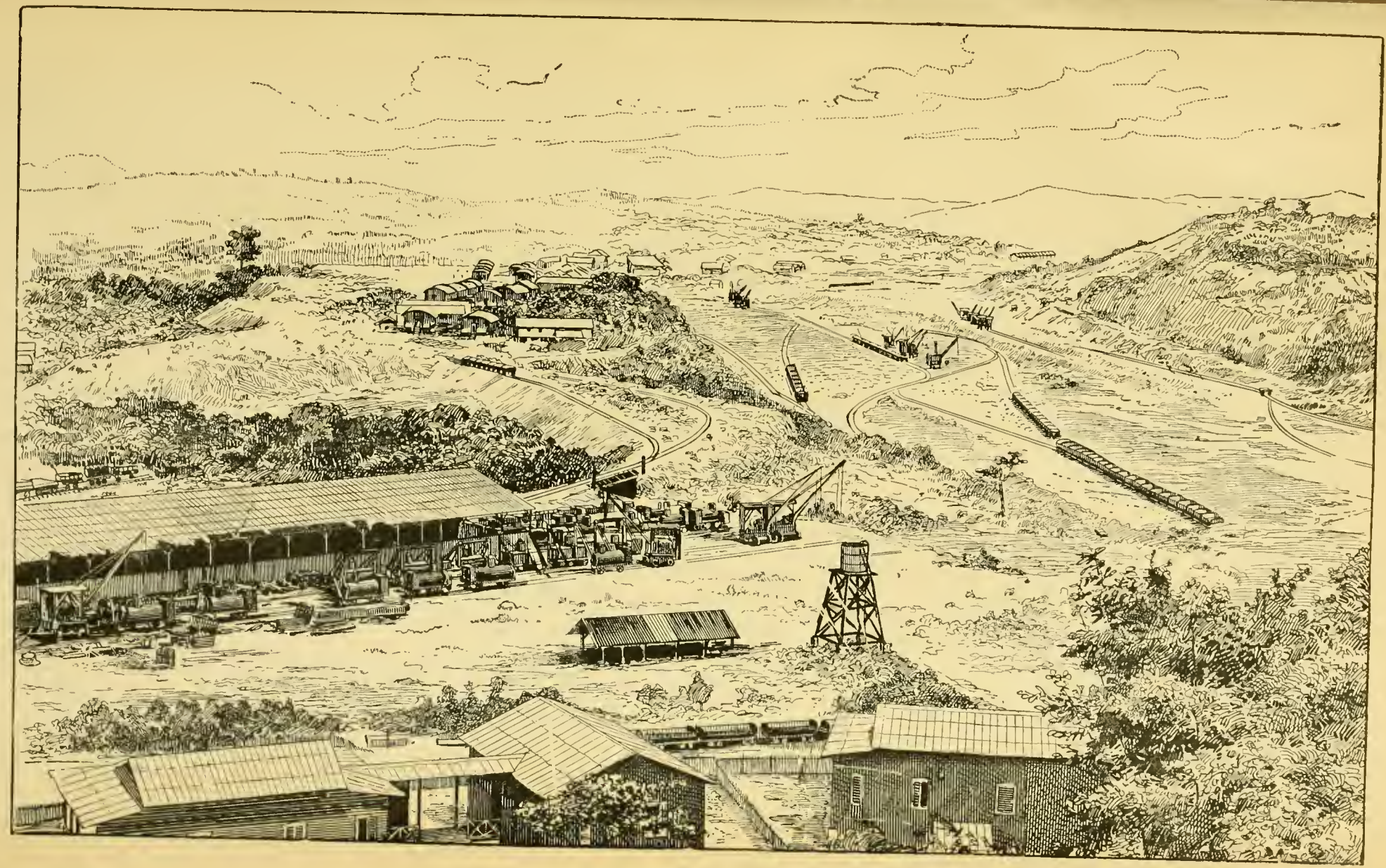


PLATE LXIV.

CUT AT EMPERADOR.

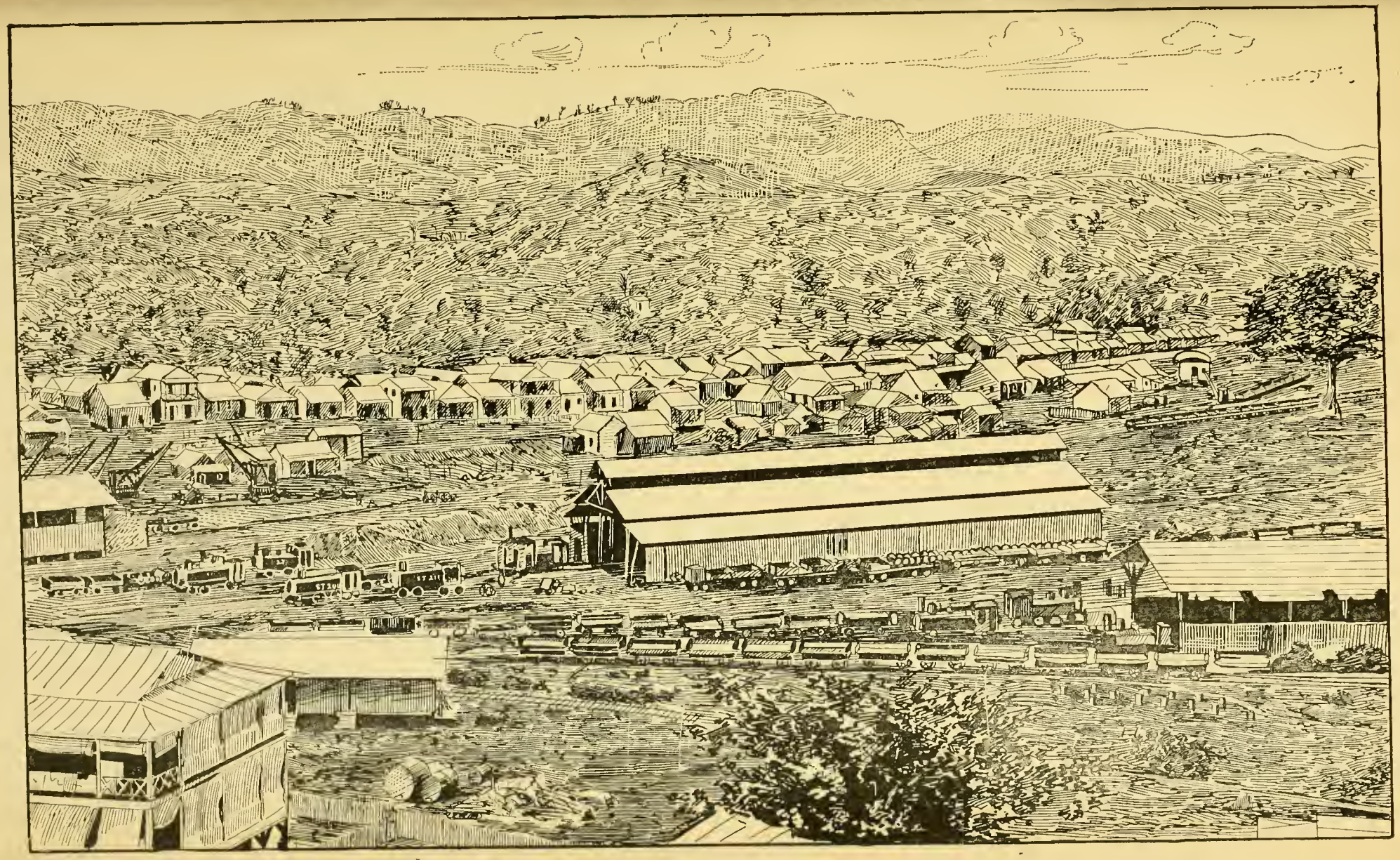


PLATE LXV. MACHINE SHOPS AT EMPERADOR.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

401.

(Vol. XIX.—December, 1888.)

NEW TRANSFER BRIDGE, HARSIMUS COVE, JERSEY CITY, N. J.

By J. A. BENSEL, Jun. Am. Soc. C. E.

READ OCTOBER 3D, 1888.

WITH DISCUSSION.

The idea of a counterbalance bridge for the transferring of cars from the shore to a float is not new, but so far as is known the bridges shown on the drawings are the only ones now in use in this harbor.

The bridges shown on the plans have been in use for over six months and are working so satisfactorily that others of the same general design are now being erected in Philadelphia.

The extra pair of trusses shown on the drawings were not originally intended, but were necessitated by the extra heavy weight of the yellow pine, and some extra weight added to the bridge, principally in the apron.

The end movement of the apron is controlled by a friction gearing, not shown on the drawings; the friction being sufficient to hold the apron in position when not attached to a float, and yet still allow the desired movement when the float is loaded or unloaded.

These bridges are placed near the north boundary of Jersey City in a place where the shoaling of the water occurs very rapidly, and pontoon bridges could not have remained in service more than a year, when it would have been necessary to have removed them in order to dredge.

Other advantages of this type are their easy adaptability to any height of float, the prevention of warping which occurs with the floating bridge when a heavy load is taken along one side without loading the other, and the fact that, the bridge being always above the water, the ice, which in the pontoon bridge is the source of considerable trouble and expense, can have no effect.

The company is now having a small engine geared to the bridge in order to raise and lower by steam power, and the requisite boilers in this case being close at hand, there will be a considerable saving in the running expenses. For the remainder the plans will explain themselves. (See Plates LXVI and LXVII.)

DISCUSSION.

J. J. R. Croes, M. Am. Soc. C. E.—What is the difference between the floats?

Mr. Bensel.—The end of the bridge is raised and lowered between its two extreme positions about 10 feet.

John Bogart, M. Am. Soc. C. E.—That is the difference between the different floats and the difference between high and low tide?

Mr. Bensel.—Yes, sir.

Theodore Cooper, M. Am. Soc. C. E.—Do you employ three men to raise and lower the bridge or do you do it by steam?

Mr. Bensel.—At the present time one man, an engineer, raises and lowers the bridges by steam, sometimes working both bridges together and at other times working them separately. The bridges are worked throughout the day, the intention being to raise and lower with the tide, and then to take up the small differences between the heights of loaded and unloaded floats and the different sizes of floats with the apron.

The apron being counterbalanced, can be raised by two men, one on each side, and is generally kept raised a little above the height of any float, and when the float comes into the slip, the apron is dropped to the proper level by means of a friction device that can be operated with one hand, the friction being just sufficient to hold the apron in any position unless the handle is held down.

Mr. Cooper.—All the hoisting is done at the sides?

Mr. Bensel.—The engine is placed on the platform between the two bridges and thus operates both bridges from the sides; a leather belt conveys the power from the engine to the horizontal shaft on top of the overhead trusses, then passing over a friction pulley, the levers from which are operated by the engineer, and throw the bridges in or out of gear.

Mr. Cooper.—When you worked it by men, how many men did it take?

Mr. Bensel.—Five men operated the two bridges, and although they were idle a considerable part of the time, the work, particularly on hot days, was severe, and it was deemed both expedient and economical to put in the steam power. The traffic at the time the bridges were worked by hand was not very heavy, and one bridge did almost all the work. Probably to work both bridges with the present traffic, and have no delays, would require six men.

The bridges being located near the grain elevator, the steam could be cheaply applied, and one small double cylinder engine, about four horse-power, serves for the two bridges. There are now employed to operate the two bridges four men.

A Member.—Do you know whether in pontoon bridges the machinery is built on this principle?

Mr. Bensel.—No; I think pontoon bridges, certainly all that I have seen about here, for the transferring of cars, depend almost entirely on the pontoon for raising and lowering. They all have a light gearing, which is intended to raise the bridge a small distance above the height given by the pontoon, when, as frequently happens, the pontoon leaks or is covered with ice; but as the bridges are without counterbalances, the distance they can be raised is very small, and they are frequently built so as to stand somewhat above any ordinary light float, and then are sunk to the proper level by running an engine down one side as far as may be necessary to slip the toggle irons in place. Besides the loss of time with this arrangement, toggle irons are frequently broken, as a loaded float is virtually hung on the toggles, and unless it is pulled out of the slip so that the toggle irons are released at the same time, something is apt to break, and toggle irons are both expensive and difficult to replace.

Mr. Bogart.—Is it very much more expensive than the old construction?

Mr. Bensel.—I would not care to give any figures as to the cost of these bridges. They were constructed by the company's men in the best possible manner and of the best material, and it would be difficult to duplicate them in a contract job. Constructed by contract they would probably cost somewhat more than the pontoon type, but I can give no figures.

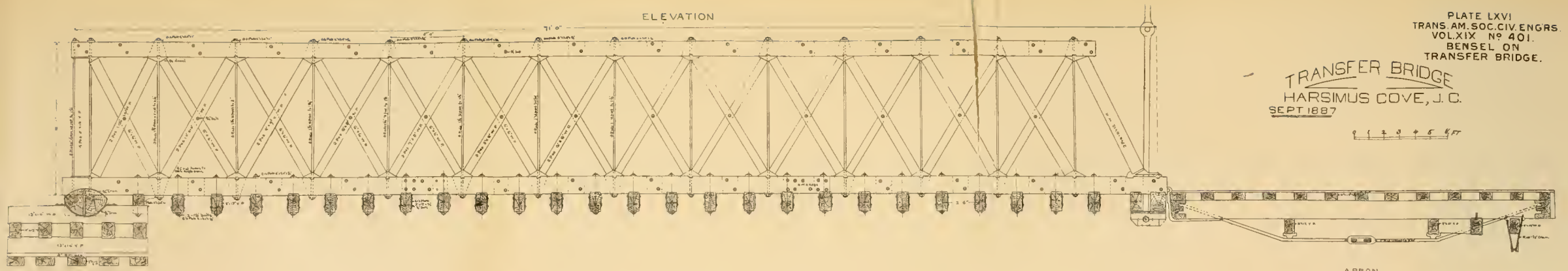
A Member.—Who designed the bridges?

Mr. Bensel.—They were designed in Jersey City by the Pennsylvania Railroad Company.

PLATE LXVI
TRANS. AM. SOC. CIV. ENGRS.
VOL. XIX Nº 401.
BENSEL ON
TRANSFER BRIDGE.

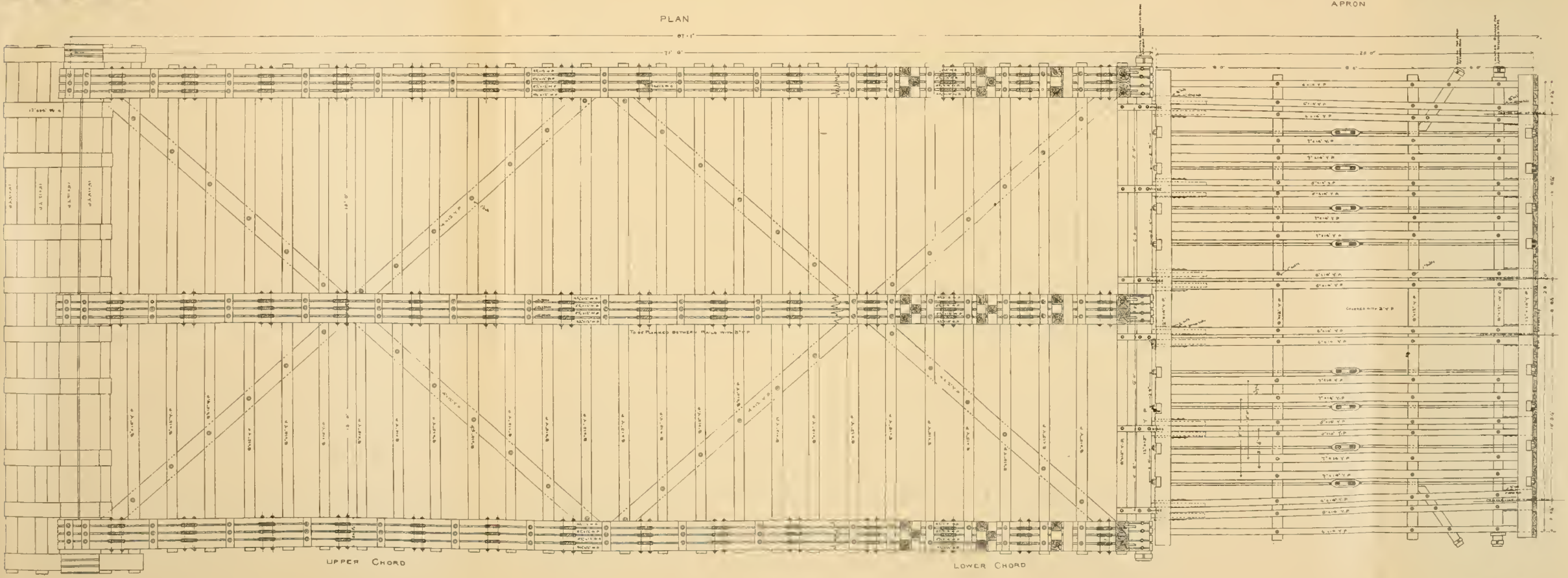
TRANSFER BRIDGE
HARSIMUS COVE, J. C.
SEPT. 1887

ELEVATION



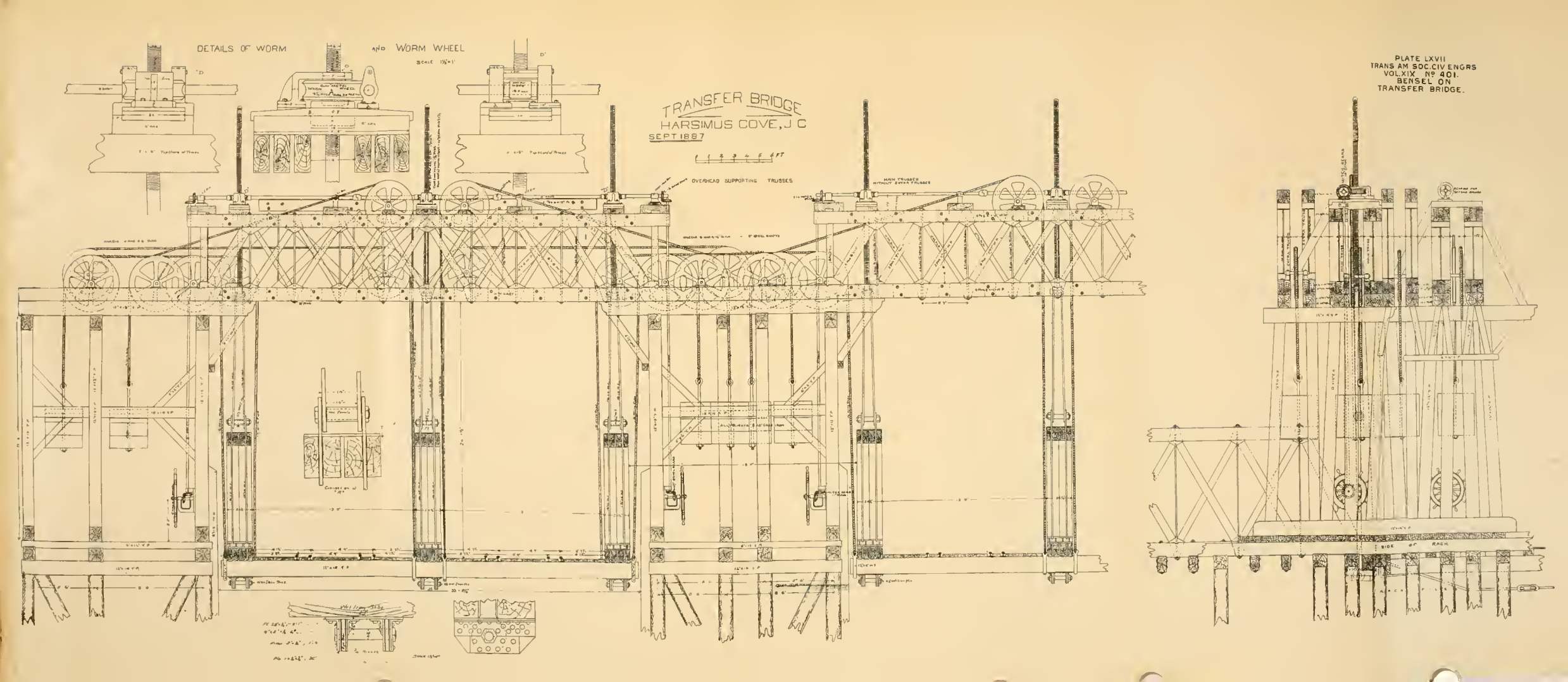
APRON

PLAN



UPPER CHORD

LOWER CHORD


DETAILS OF WORM AND WORM WHEEL
TRANSFER BRIDGE
HARSIMUS COVE, J C
SEPT 1887
OVERHEAD SUPPORTING TRUSSES
PLATE LXVII
TRANS AM SOC. CIV ENGRS
VOL. XIX Nº 401.
BENSEL ON
TRANSFER BRIDGE.

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